Breaching of sea dikes initiated by wave overtopping
A tiered and modular modelling approach

Dissertation

submitted to and approved by the
Faculty of Architecture, Civil Engineering and Environmental Sciences
University of Braunschweig – Institute of Technology

and the
Faculty of Engineering
University of Florence

in candidacy for the degree of a
Doktor-Ingenieur (Dr.-Ing.) /
Dottore di Ricerca in Risk Management on the Built Environment *)

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Submitted on 31 March 2007
Oral examination on 7 May 2007
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2007

*) Either the German or the Italian form of the title may be used.
The dissertation is published in an electronic form by the Braunschweig university library at the address

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To Gabriella, Giovanni and Paolo
Preface

The following study has been developed in three years (2004-2006) at the Dipartimento di Ingegneria Civile of University of Florence (Italy) and at Leichtweiss Institut für Wasserbau, Abteilung Hydromechanik und Küste ingenieurwesen of Technical University of Braunschweig (Germany). Basic motivation of the study is the need to improve the capability of defence systems against coastal floods. In fact, the interest of the community on flood risk, already very high, is being strongly enhanced after recent catastrophic events, like hurricane Katrina in U.S.A. (September 2005).

My best thanks to my tutors, Prof. H. Oumeraci and Prof. P.L. Aminti, that constantly supported and advised me. Both of them showed me how to do scientific work and gave an important contribution to my education. I am also extremely grateful to Dr. Andreas Kortenhaus, who continuously advised me and gave me the opportunity to take part to meetings and workshops of the European research project FLOODsite (Integrated Flood Risk Analysis and Management Methodologies). Thank you also to Mark Morris and Dr. Mohammed Hassan from HR Wallingford (UK) for the material that they kindly provided me.

I feel also grateful to Prof. Claudio Borri and Prof. Udo Peil, coordinators of the Doctoral Course, because they offer the chance to attend this programme.

Very kind thanks to my German colleagues and friends, Markus Brühl, Matthias Kudella, Juan Recio and Aga Strusinska, that made me feel part of the Institute, and specially Peter Geisenhainer and Grzegorz Stanczak for the scientific discussions that we had, their advices and cooperation. A special thank to Gabi Fournier, who helped and supported me in all practical and organisational problems at the Institute and has been my patient and efficient German teacher. Thanks to my Italian colleagues and friends, Lorenzo Cappietti, Enrica Mori and Grazia Tecchi who supported me and shared all every day research activities. A kind thank to Serena Cartei who solved for me all administrative problems.
Abstract

Sea dikes are used as defence structures against flooding in lowland areas with relatively high storm surge levels. The formation of a dike breach induced by wave overtopping was one of the most frequent causes of dike failure associated with disastrous damages. Therefore, dike breaching is closely related both to the dynamics of a protected coast and flood risk assessment/management. Despite the importance of dike breaching, the underlying processes, their simulation and prediction are still not well understood.

Based on an extensive literature study on the most relevant processes associated with dike breaching and models available, a tiered modular modelling strategy has been developed which consists in a preliminary and a detailed breach model that build an appropriate model system for engineering practice. The model system applies to sea dikes made of a sand core and a clay cover with grass.

Results from the preliminary model provide an overview of the overall breaching process and indicate which improvements are required in the detailed model. Results from the detailed model which includes new processes and model improvements removing some assumptions of the preliminary model provide a first step toward the development of a fully process-oriented breach model. Model validation against laboratory tests and experienced dike failures, although tentative, provides very encouraging results.

Model uncertainties are evaluated making use of sensitivity analysis and level III reliability analysis. The model results are associated with large uncertainties which are mainly originated by the inputs parameters, especially the material properties.

The overall results illustrate a tiered modular approach for the simulation of breach formation and growth in a real sea dike initiated by wave overtopping. The model system provides a proper departure basis for a fully process-oriented description of the breaching process in sea dikes.
Sommario

Gli argini di difesa della costa (dighe costiere) sono diffusamente impiegati come protezione dalle inondazioni in aree sotto il livello del mare o interessate da subsidenza e dove si registrano relativamente alti sovralzi da vento. La formazione nella diga di una rottura (breccia) dovuta alla tracimazione ondosa è stata una delle più frequenti cause di catastrofiche inondazioni. La formazione di breccie è pertanto legata sia alla dinamica che alla stima e alla gestione del rischio costieri. Tuttavia, la fisica del fenomeno e la sua previsione non sono state ancora completamente chiare.

Basandosi su un’estesa analisi della letteratura disponibile, è stato sviluppato un sistema di modellazione modulare a livelli applicabile anche nella pratica ingegneristica. Il sistema di modellazione è costituito da un modello preliminare e uno dettagliato e riproduce una diga con un corpo in sabbia e un rivestimento in materiale coesivo vegetato.

Il modello preliminare fornisce una panoramica del fenomeno e indica come superare nel modello dettagliato alcune delle maggiori limitazioni.

Il modello dettagliato, riducendo le ipotesi del modello preliminare, include alcuni nuovi aspetti, migliora la simulazione di altri e costituisce un primo passo verso un modello completamente orientato alla fisica del fenomeno.

La validazione dei due modelli con dati di laboratorio e casi reali di rottura, anche se preliminare, fornisce incoraggianti risultati.

Le incertezze dei due modelli sono stimate mediante analisi di sensitività e di affidabilità (Livello III). Gli output dei due modelli sono associati ad un alto livello di incertezza dovuto soprattutto agli input, specialmente alle proprietà dei materiali.

Il presente studio suggerisce un sistema di modellazione modulare a livelli per la simulazione di breccie in dighe costiere dovute alla tracimazione ondosa. Il sistema di modellazione stabilisce una base adeguata per una descrizione completamente orientata alla fisica del fenomeno.
Kurzfassung


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1. Introduction

1.1. Problem definition and motivations

The extent of densely populated coastal areas prone to flooding is very high. In Europe, countries that border the North Sea have always experienced very high flood risk compared to the other countries, but catastrophic events have been recorded all over the world, with tremendously high losses and human lives (The Netherlands, 1953; Germany, 1962; Bangladesh, 1970, 1991; Florida, USA, 2005). Moreover, water level rise due to climate changes would have important implications, including an additional increase in flood risk.

Sea dikes are among the most ancient and widely used defence structures against coastal floods. In The Netherlands, during the Middle Age, water retaining structures in form of dikes, have been realized just to protect single farms, but soon, since the 9th century, these local flood defences have been gradually extended to form a compact and unique defence system (Vollmer et al., 2001).

Until the 19th century, sea dikes were designed according to an experience-based approach. The design water level was calculated based on the highest known water level, introducing an inadequate safety level against flooding. After the flooding of 1953 in The Netherlands (D'Angremond, 2003; Pilarczyk, K.W., 1998), an overload-based approach was introduced and still used. The design water level is given by extreme storm surge levels with a certain yearly probability of exceedance, and some other structural requirements are prescribed, but different countries apply different standards (Jorissen et al., 2000) and the probability of failure of the whole defence system, which is the most important parameter in assessing and managing the flood risk (Oumeraci & Kortenhaus, 2002; Voortman et al., 2002) is in fact unknown. Evaluation of flood risk and optimisation of risk mitigation strategies only can be achieved through a flood risk-based approach of the design process, where dike failure represents the vulnerability of the flood defence and the way risk sources are transferred to receptors, i.e. risk pathway (Figure 1.1).
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The breaching process affects both flood risk assessment and management (Figure 1.2). In fact, the breaching process is important for (i) the dike failure assessment and thus for the calculation of the probability of failure of the dike and the probability of flooding (Kortenhaus and Oumeraci 2002), (ii) the estimate of the warning time (Wahl, 2004) and the effective development of emergency plans, early warning systems, evacuation measures and definition of threats to human lives and properties and (iii) the accuracy (Wahl, 2004) and initial conditions at the dike breach for flood propagation modelling, such as the outflow hydrograph and the water depth at the breach channel exit (Figure 1.2).

Dike breaching can be defined as a failure of the dike and the related uncontrolled release of water in form of wave overtopping, overflow or a combination of both (Figure 1.3a), subsequent formation of an initial breach (Figure 1.3b) and growth up to the final breach (Figure 1.3c). Prediction of breaching in sea dikes consists in the determination of "where the breach will occur, when it will occur and what will happen if it does occur" (Morris, M. & Hassan, 2002).

Concerning sea dikes, the probability of breaching is not adequately defined under the current safety standards, although it represents one of the most tangible cause of flood hazard. Failure due to breaching gives the greatest contribution to
uncertainties within the whole process of failure analysis. Moreover, it has been proved from past dike failures and fault tree analysis of real sea dikes (Kortenhaus, 2003) that one of the main causes of dike failure is the breaching induced by wave overtopping. Nevertheless, available breach models are rather qualitative tools (Lecointe, 1998; Morris, M.W., 2000). Therefore, the development of a new breach model for sea dikes is urgently needed.

Figure 1.3: Dike breaching process induced by wave overtopping (see Annex C for references)

1.2. Objective

The main aim of this study is the development of a new model for dike breaching due to wave overtopping in a real sea dike. The new model applies to sand-clay dikes with a sand core and a clay cover with grass. The breach process is simulated from its initiation to the development of the final breach. Considerations related to breach location along the dike line are neglected and the simulation domain is limited to a homogenous cross-section of the dike. Causes of breaching included in the model are wave overtopping, overflow, a combination as well as water infiltration in the dike. The model includes most of physical processes involved and provides a first-step towards a complete process-oriented simulation of dike breaching. A deterministic approach is mainly followed in this study, but uncertainties, sensitivity and reliability analysis are also provided. This might be the first basis for a future reliability approach. Results of the model include breach width and depth, outflow hydrograph and time associated with the entire breaching process.

1.3. Methodology

The overall methodology of the study is summarised in Figure 1.4, together with the related Chapters of the thesis.

In Chapter 2, a detailed review and analysis of the present knowledge on breaching is proposed. Causes of breach initiation, erosion and sediment transport processes are reviewed and a list of available breach models is provided. As a final result a detailed methodology is derived.
In Chapter 3, a preliminary breach model is developed. Details about the hydrodynamic and morphodynamic modules are explained, results from the model and a tentative validation are presented.

In Chapter 4, a detailed breach model is developed. Presented features of hydrodynamic and morphodynamic modules only concentrates on those aspects and processes that are not included in the preliminary model or that have been considerably improved. A comparison of the results obtained in the preliminary and in the detailed model and a tentative validation are finally provided.

In Chapter 5, uncertainties associated to the model system are quantified by means of sensitivity and reliability analysis.

In Chapter 6, conclusions about the proposed model system and recommendations for future research are provided.
2. Dike breaching processes and modelling: state of the art and specification of objectives and methodology

Dike breaching is a 3D process that may lead to the collapse of the dike and thus to flooding. Possible causes of breach initiation result in several failure modes leading to breaching. Concentrating on real sea dikes, first, the grass cover is removed due to sheet erosion or turf set-off, then the clay layer fails due to headcut erosion and clay cover instability, and finally the sand core is washed out.

In this Chapter, the available knowledge on breaching processes and modelling is reviewed and analysed. As a result, the requirements for a new model are derived. In particular, the following aspects are investigated:

- Analysis of causes of breaching focusing on breach initiation from the inner slope (Section 2.1);
- Review of erosion and mass instability processes, including grass, clay cover and sand core (Section 2.2);
- Availability of sediment transport models for dike breaching (Section 2.3);
- Classification and discussion on available breach models (Section 0).

As a final result of this review, the objectives, the detailed methodology and procedure of the present study are defined (Section 2.5). More details are provided in D’Eliso et al. (2005).

2.1. Causes of breach initiation

The breaching process is the consequence of morphological and hydraulic boundary conditions. Natural causes of breach initiation are (i) wave overtopping (Figure 2.1a), (ii) a combination of wave overtopping and overflow (hereafter also called combined flow) or pure overflow (Figure 2.1b), (iii) wave impact (Figure 2.1c) and (iv) infiltration and seepage (Figure 2.1d). Wave impact is not included in the study and is the subject of an on-going Ph.D. thesis by Stanczak which will be completed in 2008.
Breaching of sea dikes initiated by wave overtopping

2.1.1. Morphological and hydraulic boundary conditions

Sea dikes are water retaining earth structures. This study concentrates on typical North Sea dikes, made of a sand core and a clay cover with grass (Figure 2.2a).

Sand is often made of a loose, non-cohesive grain structure with a good permeability and a relatively large mass per unit volume (Table 2.1).

Clay is a cohesive soil, mainly made up of fine particles (TAW, 1996, 2000). Key properties of natural clay are water retaining capacity and cohesion, while permeability and erosion resistance are also determined by the formation of the so-called soil structure, which is a time-varying process (Figure 2.2b and Table 2.1).

Grass cover is grassland vegetation rooted in soil (Figure 2.2b and Table 2.1). The upper part, called turf, has high root density (≥ 1 m/cm³), is porous and elastic in moist conditions. The underlying layer, i.e. substrate, is well fixed and plastic in moist conditions. Erosion resistance offered by grass roots is concentrated in the sod, which is a structured soil, consisting in aggregates separated by pores and roots and divided in an upper part, the turf, and a lower part where root density decreases with depth (TAW, 1999, 2000).

Sea action on dikes consists in extreme water levels and waves. Short-term water level fluctuations (Kamphuis, 2000), especially storm surges, together with tides and barometric surges, are the main causes of breaching. Waves in shallow water, at the dike toe, are well represented by double peaked wave energy spectra
(Groenendijk & Van Gent, 1998), but among theoretical spectra, the TMA spectrum (Kitaigorodskii et al., 1975) is the most widely used.

Table 2.1: Typical properties of sand, clay and grass as construction materials

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Typical values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SAND</td>
</tr>
<tr>
<td>Composition</td>
<td>-</td>
<td>Silt - Clay &lt; 5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sand &lt; 25%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lutum &lt; 20÷50%</td>
</tr>
<tr>
<td>Density of soil</td>
<td>kg/m$^3$</td>
<td>1900-2100</td>
</tr>
<tr>
<td>Sediment size</td>
<td>mm</td>
<td>0.1-0.5</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td>m/s</td>
<td>$10^{-2}$-$10^{-2}$</td>
</tr>
<tr>
<td>Angle of friction</td>
<td>deg</td>
<td>30-38</td>
</tr>
<tr>
<td>Cohesion</td>
<td>kN/m$^2$</td>
<td>-</td>
</tr>
</tbody>
</table>

2.1.2. Wave overtopping and overflow

Wave overtopping and overflow at a sea structure are normally evaluated with the associated mean discharge (Oumeraci et al., 1999; TAW, 2000; Van der Meer, 2002). However, a mean value is not sufficient to assess the erosion. Both flow velocity ($v$) and flow depth ($h$) along the dike profile are needed.

There are two possibilities to calculate wave overtopping and overflow at the dike (D'Eliso et al., 2005; Morris, M. & Hassan, 2002):

a) Empirical formulae and steady non-uniform free surface flow equations

- Wave overtopping: two sets of formulae have been derived based on laboratory tests (Schüttrumpf & Oumeraci, 2005; Van Gent, 2001). Starting from wave run-up, flow velocity and depth along the dike profile are calculated. Results of the two sets of formulae do not show significant differences (Van Gent, 2002);
- Overflow: the overflow discharge is calculated with weir formulae, e.g. Poleni formula (Oumeraci et al., 1999), flow is assumed critical at the dike crest and steady non-uniform flow equations are applied at the inner slope;
- Combined flow: similarly to overflow, a modified weir formula (Bleck et al., 2000) and steady non-uniform flow equations are used to calculate the combined flow discharge and flow.

b) Numerical models

- Saint Venant’s equations for overflow;
- Shallow water equations: 1D-ODIFLOCS Model (Van Gent, 1995), 2D-OTT Model (Clarke et al., 2004) for wave overtopping and DEICH_2D Model (Broich, 2004) for overflow;
- Reynolds-averaged Navier Stokes equations (RANS-VOF Models): 2D Model COBRAS (Liu & Lin, 1997), 2D Model of Soliman (Soliman et al.,
Breaching of sea dikes initiated by wave overtopping and 2D Model of Busnelli (Busnelli, 2001) for overflow.

Empirical formulae, steady non-uniform flow and Saint Venant’s equations are 1D models, shallow water equations are depth-averaged models and RANS-VOF models are 2D process-oriented models, but the computational effort may be very high. RANS-VOF Models give a more accurate description of flows with steep water level gradients and mixed flow conditions than shallow water equations (Busnelli, 1999, 2001). RANS-VOF models are at present the most efficient tool for overtopping flow during dike breaching. Moreover, they can easily be extended to describe 3D flow if required.

2.1.3. Infiltration

Water infiltration and seepage flow might occur due to three main factors (Figure 2.1): (i) mean water level, (ii) waves (wave overtopping and run-up and run-down), (iii) rain. The relevance and effects of infiltration on dike breaching are summarised in Table 2.2.

Table 2.2: Relevance of the infiltration processes for dike breaching

<table>
<thead>
<tr>
<th>Cause</th>
<th>Mean water level</th>
<th>Wave overtopping and overflow</th>
<th>Wave run-up and run-down</th>
<th>Rain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>seaside</td>
<td>crest and landside</td>
<td>seaside</td>
<td>overall</td>
</tr>
<tr>
<td>Infiltration</td>
<td>high phreatic line and seepage</td>
<td>high water content and local saturation</td>
<td>local saturation</td>
<td>high water content</td>
</tr>
<tr>
<td>Main direction</td>
<td>horizontal</td>
<td>vertical</td>
<td>horizontal</td>
<td>vertical</td>
</tr>
<tr>
<td>Total time</td>
<td>hours</td>
<td>hours</td>
<td>minutes</td>
<td>days</td>
</tr>
<tr>
<td>Time of an event</td>
<td>-</td>
<td>seconds</td>
<td>seconds</td>
<td>-</td>
</tr>
<tr>
<td>Mass instability</td>
<td>inner slope and breach slopes</td>
<td>grass and clay cover</td>
<td>negligible</td>
<td>negligible for sea dikes</td>
</tr>
</tbody>
</table>

The influence of wave run-up and run-down and of rain is of secondary importance and is therefore neglected in this study. The influence of wave-induced set-up can be also neglected because it locally affects the dike (Massel, 2001). Infiltration and seepage flow induced by high mean water levels depends on the presence of cracks at the outer slope and mainly proceed horizontally, while infiltration due to wave overtopping and overflow is almost vertical and contributes to saturate the clay cover close to the surface. Under the superficial saturated area, a more extended unsaturated zone develops. The infiltration rate is higher at the dike crest, where the erosion is negligible, than at the inner slope (Möller et al., 2002).

There are two levels of detail in predicting the infiltration induced by wave overtopping and overflow:

- **Simplified models**: sets of equations, based on laboratory tests and simplified solutions of Richard’s equation, to calculate the saturated ($z_s$)
and infiltration \((z_w)\) water fronts in the dike (Wang et al., 2003; Wang, 2000a; Weißmann, 2003);

- **Finite element models:** numerical solution of Richard’s equation, which is valid in saturated and unsaturated conditions and combines Darcy’s law with water conservation through the porous medium, i.e. continuity condition (Richards, 1931).

Finite elements models provide a more complete description of infiltration.

### 2.1.4. Implications for the present study

Hydrodynamic models for wave overtopping, overflow and infiltration due to them are normally 1D or 2D. Therefore, they can only provide inputs for 1D or 2D erosion and sediment transport models. Moreover, free surface flow models and infiltration models are normally not coupled.

Simple models reviewed in Sections 2.1.2 and 2.1.3 are very practical to use and recommended if the computational effort must be kept low, as for preliminary calculations and uncertainty analyses (Chapters 3 and 5). Particularly concerning infiltration, there is no universally accepted model and a selection of the most appropriate among the existing models is recommended.

### 2.2. Erosion and mass instability processes and models

Dike breaching is the result of a series of physical processes of erosion and mass instability that occur simultaneously or as a sequence of events, partially similar to breaching of homogenous non-cohesive and cohesive dams (man-made or landslide), earth spillways, fuse-plug dams, natural coastal barriers and dunes. Eight main processes are identified and reviewed (Figure 2.3):

- Grass erosion (Figure 2.3a and Section 2.2.2);
- Clay erosion, headcut erosion and advance in the clay cover, headcut in sand-clay scour (Figure 2.3c, d, e and Section 2.2.3);
- Sand erosion, breach slopes instability (Figure 2.3g, h and Section 2.2.4);
- Grass and clay cover instability (Figure 2.3b, f and Section 2.2.5).

These processes may be summarised to provide six breaching phases and associated times (Figure 2.4). Time of breaching \((t_b)\) is the result of (i) time of breach initiation \((t_i)\) that includes the failure of the grass and clay cover at the inner slope and at the crest, (ii) time of breach formation \((t_f)\) that is the time of sand core erosion and (iii) time of breach development \((t_d)\) that consists in the breach channel widening up to the final breach.

### 2.2.1. Breach parameters and flow

Dike breaching is a 3D space-time dependent process described by breach parameters and flow (Figure 2.4).
<table>
<thead>
<tr>
<th><strong>GRASS COVER</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Grass erosion</td>
<td>b) Turf set-off</td>
</tr>
<tr>
<td><img src="image1" alt="Grass erosion" /></td>
<td><img src="image2" alt="Turf set-off" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>CLAY COVER</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>c) Clay and headcut erosion</td>
<td>d) Headcut advance (GWK, 2002)</td>
</tr>
<tr>
<td><img src="image3" alt="Clay and headcut erosion" /></td>
<td><img src="image4" alt="Headcut advance" /></td>
</tr>
</tbody>
</table>

| e) Headcut in sand-clay scour | f) Clay cover sliding or up-lift |
| ![Headcut in sand-clay scour](image5) | ![Clay cover sliding or up-lift](image6) |

<table>
<thead>
<tr>
<th><strong>SAND CORE</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>g) Sand erosion</td>
<td>h) Breach slopes instability</td>
</tr>
<tr>
<td><img src="image7" alt="Sand erosion" /></td>
<td><img src="image8" alt="Breach slopes instability" /></td>
</tr>
</tbody>
</table>

Figure 2.3: Overview of main erosion and mass instability processes during breaching (see Annex C for references)
Breach location along the dike line is still an unsolved issue, due to non-uniformities in the geometry, sea exposure and constitutive materials, such as high surges and waves, steep bathymetry in front of the dike, non-uniform grass cover, dike sections of strongly structured soil, and variations in dike height.

Initial breach is usually assumed both for cohesive and non-cohesive materials and breach initiation is not really predicted. Definitions of the initial breach in the clay cover include (Section 2.2.3): (i) formation of first gully (Möller et al., 2002), (ii) scour hole as the headcut starts migrating (Hanson & Temple, 2001a) and (iii) vertical headcut with a threshold height (Elliot & Laflen, 1993). The initial breach in the sand core is the channel resulting after cover failure, with sand bed and clay vertical slopes (Section 2.2.4).

Three flow regions with different flow regimes (Figure 2.5a) are generally observed during dike breaching (Powledge et al., 1989a, 1989b):

- **Seaside:** sub-critical flow \((Fr < 1)\);
- **Dike crest:** flow develops to a critical state \((Fr = 1)\);
- **Landside:** super-critical flow and again critical flow at dike toe \((Fr > 1)\).

Flow velocity \((v)\), depth \((h)\) and discharge over the dike \((q)\) and through the breach \((Q_b)\) represent the main parameters to describe the flow field. Outflow hydrograph \((Q_b(t))\) is very low during breach initiation, then increases with time and, after the peak discharge \((Q_{bp})\), decreases according to the mean water level (MWL) and backwater effects \((h_p)\) on the outflow (Figure 2.5b). In the presence of waves, the outflow hydrograph is the result of wave-induced flow and overflow (Figure 2.5b). Breach flow is calculated with two models (Section 2.1.2):

- **Wave overtopping model** from first overtopping \((t_0)\) to threshold time \((t_t)\), when the driving breach mechanisms becomes combined flow;
- Combined flow model from threshold time \( t_t \) to final breach \( t_b \).

**Figure 2.5: Flow regimes over the dike and through the breach and breach outflow**

### 2.2.2. Erosion of the grass cover

Grass provide substantial protection against erosion (Smith et al., 1994; Temple et al., 2003), depending on root density \( \rho_r \), stem length \( L_s \) and grass conditions (green or dormant). The erosion process starts with wash-out of small soil particles at the inner slope and is dominated by particle detachment induced by the tractive stress exerted by the flow (*sheet erosion*). Once the turf has been eroded, the flow takes it off leading to the grass failure. Grass erosion is concentrated in weak sections, e.g. local settlements, small holes and grass spots. Field tests show that the erosion at the dike crest is much lower than at the inner slope and assumed negligible (Hanson & Temple, 2001a).

Available models follow four approaches (Samani & Kouwen, 2002): (i) maximum permissible velocity, (ii) maximum flow depth, (iii) equivalent stone size, (iv) permissible tractive force (effective shear stress). Only the fourth approach directly includes effects of grass and cumulative flow action and is recommended in case of wave overtopping flow. Erosion and failure are calculated as function of the effective bottom shear stress \( \tau_{0,e} \), which includes the grass cover through the grass cover factor \( C_f \) and the ratio between total Manning’s roughness \( n_{toe} \) and soil Manning’s roughness \( n_e \) (Temple et al., 1987).

### 2.2.3. Erosion of the clay cover

Physical tests on clay layer without grass, under overflow (Hahn et al., 2000; Hanson & Temple, 2001b; Hanson et al., 1999b; Temple & Hanson, 1998) and wave overtopping conditions (Möller et al., 2002) show similar erosion mechanisms. Erosion starts with wash-out of soil particles (*sheet erosion*). The surface becomes more and more rough and small holes and elevations appear (Figure 2.6a). These discontinuities are the initial condition of the erosion process, which concentrates in little channels (*micro-rills and rills erosion*), according to the theory of channel network initiation (Hahn et al., 2000). Rills develop into
deeper scour holes (master rills or gullies) and the inner slope takes a graded appearance. One main gully dominates among the others and takes the shape of an almost vertical headcut that starts migrating seaward (headcut erosion in clay and in sand-clay scour). The erosion process may schematically be described in two main phases: (i) local clay erosion due to concentrated flow in rills (Figure 2.6b) and (ii) headcut erosion due to the impinging jet at the headcut base (Figure 2.6c).

**Figure 2.6: Erosion of clay cover**

**Local clay erosion** is dominated by particle detachment induced by the tractive stress exerted by the flow.

Most of available models calculate the erosion rate as function of (i) the *excess shear stress*, which is the difference between bottom shear stress ($\tau_0$) and critical shear stress ($\tau_{0,cr}$), and (ii) the *erodibility of soil* $k_d$ (Meyer, 1964). They strongly depend on water content ($w$) and soil density ($\rho_c$) and are related to each other by an inverse relation (Hanson et al., 1999a; Wan & Fell, 2004).

**Headcut erosion** starts gradually, although it is simpler to assume a critical value of the initial headcut height $H_{H,0}$ (Elliot & Laflen, 1993). Depending on the impinging jet (Robinson et al., 1999), on soil properties and on advance modes (Stein & Julien, 1993), headcut erosion is the result of four erosion mechanisms (Figure 2.6c):

- **Scour erosion** ($dz$): erosion at the headcut base;
- **Deposition** landward the headcut (often neglected);
- **Erosion at the vertical headcut face** (often a secondary effect);
- **Headcut advance** ($dX$): cyclical mass instability from the vertical headcut face due to scour erosion.

Available models are 1D or 2D, although headcut erosion is a 3D process and are too sensitive to one or more empirical coefficient (Barfield et al., 1991; De Ploey, 1989; Elliot & Laflen, 1993; Hanson et al., 2001; Jia et al., 2002; Kohl et al., 1988; Robinson & Hanson, 1994; Temple, 1992; Temple & Hanson, 1994; Temple & Moore, 1997; Wu, W. et al., 1999). Often mass instability is not included, and the headcut advance is calculated with a continuous averaged
approach. At present, there is no universally accepted model. Most of the models are based on one of the following two approaches (Wahl, 2004):

- **Energy-based approach:** (i) scour erosion is function of the shear stress pattern and (ii) the advance rate is function of the energy dissipation or of the flow energy at the base of the headcut (De Ploey, 1989; Elliot & Laflen, 1993). These are generally continuous averaged models;
- **Shear stress approach:** scour erosion and advance rates are function of the shear stress pattern. These can be both continuous averaged models (Wu, W. et al., 1999) or discrete models (Robinson & Hanson, 1996).

### 2.2.4. Wash-out of the sand core


The erosion process starts at the inner slope and proceeds first vertically and at a lesser extent laterally. When the breach bottom reaches the dike base, i.e. full breach, lateral erosion becomes dominant, but at a decreasing rate up to the equilibrium final breach, that corresponds to a backwater level ($h_b$) equal to the mean sea level (MWL). The initial breach has a triangular shape (Figure 2.7a), but soon after breach initiation, becomes approximately rectangular (Figure 2.7b), with almost vertical slopes $\gamma$ (Rozov & Chanson, 2005). Final breach is more trapezoidal or parabolic (Figure 2.7c), with slopes angle equal to angle of repose of the sand $\varphi_{0s}$ (Coleman et al., 1997; Hassan et al., 1999).

![Figure 2.7: Wash-out of the sand core](image)

Depending on the sediment sizes, the water content and the compaction level (IMPACT, 2004; Tinney & Hsu, 1961), wash-out of sand core is the result of two erosion mechanisms (Broich, 2004; Hassan et al., 1999; Rozov & Chanson, 2005), see Figure 2.7d:

- **Scour erosion:** erosion of breach concentrated below the water level (vertical $dz$ and lateral $db$);
- **Breach slopes instability:** cyclical instability of breach slopes due to the scour erosion.
As a result of the breach slopes instability, the breach channel is totally or partially closed. The erosion stops or slows, until the wasted sediments is washed away according to the transport capacity of the flow, but is normally removed quite rapidly (Broich, 2004; Hassan et al., 1999).

Most of the available models are derived (i) for homogeneous dams, which are normally shorter than sea dikes, (ii) in case of overflow, (iii) neglect breach slopes instability, applying a continuous averaged approach and (iv) starts with an assumed initial breach channel (Table 2.3).

Table 2.3: Some recent conceptual sand erosion models

<table>
<thead>
<tr>
<th>Model reference</th>
<th>Structure</th>
<th>Approach</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visser (1998b)</td>
<td>Sand dike</td>
<td>Continuous</td>
<td>5 stages model</td>
</tr>
<tr>
<td>Hassan (2002)</td>
<td>Dam</td>
<td>Discrete</td>
<td>Small scale tests</td>
</tr>
<tr>
<td>Coleman et al. (2002)</td>
<td>Dam</td>
<td>Continuous</td>
<td>Empirical</td>
</tr>
<tr>
<td>Rozov &amp; Chanson (2005)</td>
<td>Dam</td>
<td>Continuous</td>
<td>Small scale tests</td>
</tr>
<tr>
<td>Rozov (2003)</td>
<td>Dam</td>
<td>Continuous</td>
<td>Analytical</td>
</tr>
<tr>
<td>Verheij (2003)</td>
<td>Dam</td>
<td>Continuous</td>
<td>New erosion formula</td>
</tr>
</tbody>
</table>

2.2.5. Mass instability due to infiltration

The **instability of grass and clay cover** are of two types (TAW, 2000):

- **Grass set-off and cover sliding**: as a result of infiltration due to wave overtopping and overflow, the weight of the soil increases and the shear stress equals the shear strength;

- **Cover up-lift and cover push-off**: as a result of infiltration due to (i) wave overtopping and overflow and (ii) seepage, water infiltrated in the core cannot be easily expelled. The resulting water pressure at sand-clay interface tends to up-lift or push-off the cover. Cover push-off, which is not induced by wave overtopping and overflow, is neglected in this study.

Vegetated clay layers are more exposed to cover instability than non-vegetated clay layers, because the surface is more erosion-resistant and the infiltration rate is higher due to the enhanced permeability induced by grass roots (Möller et al., 2002).

The instability of the vegetated clay cover with grass has not been reproduced yet in laboratory tests, but often observed during experienced dike failures. Available breach models neglect infiltration and don’t simulate clay cover with grass (Section 2.4).

There are two approaches to calculate grass set-off, cover sliding and up-lift:

- **Limit equilibrium methods**: failure surface is assumed parallel to the inner slope, at the saturated water front or at sand-clay interface (Wang, 2000b; Weißmann, 2003);
- Finite element methods: failure surface is not assumed, but calculated and the model is more process-oriented (Weißmann, 2003).

The influence of the grass cover on the stability of the cover layer is quantified through the grass root cohesion \(c_g\) which is added to the clay soil cohesion \(c_c\) (Michalowski & Zhao, 1996; Styczen & Morgan, 1995; Wang, 2000b; Young, 2005).

The instability of the inner slope, such as sliding, up-lift and heave (TAW, 2000), due to (i) very steep slopes and (ii) high phreatic line, has been reproduced during laboratory tests on breaching induced by overflow (Tingsanchali & Hoai, 1993), but its influence on dike breaching is neglected in this study, because it is not caused by wave overtopping and overflow. Few of available breach model include sliding of the inner slope (Fread, 1988; Tingsanchali & Hoai, 1993).

2.2.6. Implications for the present study

Analysing the results of erosion and mass instability during dike breaching, the following list of reasonable assumptions and limitations of available models has been derived:

(i) Erosion at the crest is much lower than at the inner slope and can therefore be neglected;
(ii) Breach initiation models are not available;
(iii) Headcut initiation models are not available;
(iv) Most headcut models are continuous;
(v) When failed at the inner slope, the remaining clay cover doesn’t affect the erosion of the sand core;
(vi) Breach cross-section has almost vertical lateral slopes;
(vii) Breach channel has an hourglass shape at equilibrium;
(viii) Most sand wash-out models are continuous;
(ix) Although not immediately removed from the breach channel, the wasted sediments is rapidly washed away from the breach channel;
(x) Erosion of the dike base is often neglected;
(xi) Influence of the infiltration on the erosion is generally neglected;

2.3. Sediment transport processes and models

Eroded sediments, both in the clay cover and the sand core are removed by the flow, as bed and suspended load. The sediment transport processes associated with dike breaching are generally out of the applicability ranges of available sediment transport models, thus limiting their capability of the latter when applied in breach models.
2.3.1. Sediment transport processes

During breaching, the transport of sediments is essentially determined by the transport capacity of the flow. The transport process develops under extreme flow conditions:

- Both bed and suspended sediment transport are important. Bed load is responsible for breach bottom evolution (Broich, 2004), while stronger flow and finer sediments result in high sediment concentrations;
- Steep slopes (inner slope $\beta$ and breach slopes $\gamma$): $\beta \approx 30^\circ$; $\gamma > \varphi$ ($\varphi \approx 32^\circ$);
- Unsteady transitional flow (Section 2.2.1), with highly super-critical flow at the inner slope: Froude number $Fr = 1/5$ and Shields parameter $\theta = 0.3/100$;
- Water-sediment mixture: $C_s \approx 0.25$ and $z_0 \approx 3D_{50} + O(h)$;
- Water viscosity ($\nu$) is larger than viscosity of pure water (Van Rijn, 1993);
- Sediments are transported by waves and combined waves and currents;
- Non-equilibrium sediment transport (Visser, 1998b)

2.3.2. Sediment transport models

A list of available sediment transport models, their applicability ranges and whether they have been applied in breach models, can be found in D'Eliso et al. (2005) and Visser (1998b). Available models generally fail to describe properly sediment transport associated with dike breaching (Table 2.4), due to a number of reasons:

- Models are validated on open channels and sea, but not during breaching;
- Models are usually derived for steady sub-critical flows, on mild slopes and in case of equilibrium sediment transport;
- Effect of gravity on particle motion is normally neglected or included, but at low shear stresses, up to $4 \cdot 10^{-5} \tau_{0,cr}$ (Bagnold, 1956; Kovacs & Parker, 1994; Parker et al., 2003; Seminara et al., 2002), even if, due to vertical breach slopes, it plays an important role;
- Sediment bed is often described by median size ($D_{50}$), i.e. single-grain size functions. Most of existing multiple-grain size functions cannot properly reproduce bed gradation; extensions of single-grain size functions to multiple-grain size functions may produce unreliable results;
- Prediction capability is poor in presence of combined waves and current. It is not yet clear whether and how much wave component is important;
- Models are derived for a flow rate of water-sediment mixture ($Q_{sw,b}$) much lower than the flow rate of pure water ($Q_b$), while during breaching they are comparable. Application of models derived for debris-flow do not provide better results (Tingsanchali & Hoai, 1993).
Table 2.4: Verification of sediment transport models during dike breaching

<table>
<thead>
<tr>
<th>Formula reference/Formula</th>
<th>Erosion</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyer-Peter &amp; Muller (1948)</td>
<td>Slower</td>
<td>Tingsanchali &amp; Hoai (1993)</td>
</tr>
<tr>
<td>Engelund &amp; Hansen (1967)</td>
<td>Good (last phases)</td>
<td>Visser (1998b)</td>
</tr>
<tr>
<td>Bailard (1981)/Bagnold-Bailard</td>
<td>Faster</td>
<td>Visser (1998b)</td>
</tr>
<tr>
<td>Visser (1988)/Bagnold-Visser</td>
<td>Good if limited</td>
<td>Visser (1998b)</td>
</tr>
</tbody>
</table>

Available breach models generally apply very simplified sediment transport models:

- Normally, uniform sediment transport models are used;
- In case of predefined erosion shape, e.g. constant, linear, calculation of sediment transport is not necessary;
- Grass and clay cover: an excess shear stress formula (Meyer, 1964) is used in all models;
- Sand core: often bed load is neglected (Visser, 1998b).

2.3.3. Implications for the present study

Due to strong uncertainties in the sediment transport rate, new breach models must include a set of sediment transport models (Table 2.5). Depending on the specific situation, the most appropriate model should then be selected. The total sediment transport rate should be always calculated as the sum of bed and suspended load, instead of extending a bed load model to a total transport model.

Table 2.5: Selection of sediment transport models for breaching simulation

<table>
<thead>
<tr>
<th>Formula reference/Formula</th>
<th>Transport mode</th>
<th>Model type</th>
<th>Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyer (1964)</td>
<td>Total (clay)</td>
<td>Equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Yang (1979)</td>
<td>Total (sand)</td>
<td>Equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Bailard (1981)/Bagnold-Bailard</td>
<td>Bed and suspended (sand)</td>
<td>Equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Galapatti (1983)</td>
<td>Total (sand)</td>
<td>Non-equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Smart (1984)</td>
<td>Bed (sand)</td>
<td>Equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Visser (1988)/Bagnold-Visser</td>
<td>Bed and suspended (sand)</td>
<td>Equilibrium</td>
<td>Current</td>
</tr>
<tr>
<td>Jia (2001)</td>
<td>Total (sand)</td>
<td>Non-equilibrium</td>
<td>Current</td>
</tr>
</tbody>
</table>
2.4. Available breach models

In the last decades, especially in the 80’s and 90’s, several models for dam breaching have been developed, but much minor effort has been spent on breaching of sea dikes. Despite the large number of models available, all have common limitations and poor improvements have been achieved in their prediction capability. The main reasons are: (i) limited knowledge of physical processes, (ii) rare cooperation between different areas of research involved, i.e. hydraulics, geotechnics, soil mechanics, (iii) problems in collecting data from experienced dike failures and from large-scale laboratory tests, (iv) models are often calibrated on few data and are not process-oriented. Existing models cannot be applied to dike breaching initiated by wave overtopping, but used as a base for new model development.

2.4.1. Classification of available breach models

Available models can be classified based on their physical background and on level of detail in describing all processes involved:

1. **Non-physically based** (Froehlich, 1995; MacDonald & Landgridge-Monopolis, 1984)
   - Models are based on observed and measured data, on past failure events and case studies (empirical models, case study analysis);
   - Peak outflow discharge is estimated, while a reasonable shape of the hydrograph is assumed. Eventually final breach width can be obtained;
   - Breach parameters are estimated from predictor equations, derived as best fit of available data or simply compared with similar experienced dike failures (comparative analysis, experience based models);
   - No need of computer programmes to be solved;
   - Main weaknesses are the descriptive analysis of the process and the lack of information on outflow and breach parameters over time. Moreover, data on which the models are based are affected by strong uncertainties.

2. **Semi-physically based** (Singh & Scarlatos, 1988; Walder & O’Connor, 1997)
   - Observed and measured data as well as a semi-physical description of the process generally build the model basis;
   - Outflow hydrograph and breach growth can be predicted for a pre-defined potential scenario (not necessarily observed or realistic);
   - The breaching process is predicted by combining principles of hydraulic for breach outflow and assumed rates of breach growth, e.g. linearly, time-dependent. Often, time of breaching and final breach are not calculated, but are assumed as model parameters (parametric approach and equations);
   - Simple computer programmes are required;
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- Main weaknesses are the use of breach parameters as input, while they should be calculated, and non-physically based assumptions without any effective guidance to the user.

3. Physically based (Broich, 2004; Fread, 1988; Hassan et al., 2002)
- Models are derived from physical understanding of processes;
- Overflow hydrograph and breach parameters are predicted for a realistic scenario, but simplifying the process through assumptions. Model uncertainties are quantified with different techniques (sensitivity and reliability analysis);
- The breaching process is predicted by combining principle of hydraulics, sediment transport and soil mechanics;
- Computer programmes are required;
- Main weaknesses are: (i) morphodynamic models are often 1D, (ii) sediment transport models are not valid for breaching (Section 2.3.2) and (iii) soil mechanics models are very simplistic.

4. Process-oriented
- Models are derived from a thorough physical understanding of processes;
- Models are also oriented to the description of the process without assumptions that modify the physics, e.g. a continuous averaged erosion model for core erosion is physically-based, but not process-oriented, because breach slopes instability is not explicitly included;
- Sophisticated computer programmes are required;
- Available models are not fully process-oriented.

Existing models are more indicative rather than design tools (Morris and Hassan 2002). The only available model for breaching of sea dikes is the BRES Model (Visser, 1998b), but the cause of breach initiation is pure overflow and the
dike is an homogeneous sand dike. Most of other models apply to non-cohesive homogeneous dams, rarely including cohesive dams, as the HR Breach Model (Hassan, 2002) and earth spillways with grass, as the SITES Model (NRCS, 1997). Model for dam breaching and earth spillways cannot be directly applied to sea dikes because the physical processes are different, i.e. (i) sea waves as primary load and (ii) sand-clay dikes with a protective layer of grass. A more detailed description of all models can be found in Annex A and in D’Eliso et al. (2005).

2.4.2. Limitations and uncertainties of available breach models

Ten main limitations of available models have been identified:

(i) **Cause of initiation**: overflow or piping (important for river and estuarine dikes).

   In case of sea dikes major cause of breach initiation is wave action;

(ii) **Type of structure**: homogeneous dams or fuse plug dams, normally without grass cover.

   Layout and size of a typical North Sea dike are quite different.

(iii) **Breach location and initiation**: usually neglected. An initial breach is arbitrarily assumed.

   Time of breaching and warning time are strongly dependent on the initial breach and are not really predicted.

(iv) **Infiltration processes**: neglected in all available models.

   Infiltration processes influences erosion and induce mass instability.

(v) **Discrete erosion processes**: often solved with continuous averaged models.

   Process-oriented models should include discrete models;

(vi) **Breach morphology**: often assumed and not calculated (1D model).

   Development of at least 2D morphological model is required.

(vii) **Dike base erosion**: usually neglected.

   Erosion of dike base is important for the final breach.

(viii) **Backwater effects**: usually neglected or oversimplified.

   The shape of the outflow hydrograph is strongly affected by the water level growth in the inundated area.

(ix) **Validation**: models are often validated on limited data sets.

   Laboratory and field tests on real sea dikes are not available yet.

(x) **Further basic research** on erosion of cohesive materials, sediment transport models, hydraulic properties of water-sediment mixture flows, probabilistic erosion models is required.

   Model uncertainties depend on sea and material parameters as well as model parameters. Sea and material parameters normally have high standard deviations (Kortenhaus, 2003) and produce high uncertainties in the model outcomes. Uncertainty analysis of breach parameters from a database of 108 dam failures (Wahl, 2004), sensitivity analysis (IMPACT, 2004), Monte Carlo simulations
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(IMPACT, 2004) and Latin Hypercube Sampling simulations (Hodák & Jandora, 2004) indicate the importance of quantifying uncertainties, when dike breaching is simulated with deterministic models.

### 2.4.3. Implications for the present study

Available breach models cannot be directly extended to real sea dikes. Nevertheless, some parts of them are useful for the new model (Table 2.6).

<table>
<thead>
<tr>
<th>AVAILABLE MODEL</th>
<th>STRUCTURE</th>
<th>USEFUL PART</th>
</tr>
</thead>
<tbody>
<tr>
<td>SITES (NRCS, 1997)</td>
<td>Vegetated earth spillways</td>
<td>Grass and clay cover erosion</td>
</tr>
<tr>
<td>HR Breach (Hassan, 2002)</td>
<td>Non-cohesive dams</td>
<td>Core erosion</td>
</tr>
<tr>
<td>BRES Model (Visser, 1998b)</td>
<td>Sand dikes</td>
<td>Core erosion</td>
</tr>
</tbody>
</table>

### 2.5. Specification of objectives and methodology

As a main result of the review and analysis of the present knowledge and models related to breaching, the following objectives and methodology have been derived for the present study.

### 2.5.1. Objectives

Catastrophic floods in coastal zones are often due to breaching of dikes, e.g. storm surge of 1953 in The Netherlands and 1962 in Germany (Kortenhaus and Oumeraci 2002). A detailed knowledge of physical processes involved and a reliable prediction of dike breaching is then closely related to risk assessment and management of coastal floods (Figure 1.2). At present however, available breach models are not process-oriented, have not been developed for real sea dikes and waves as a cause of breaching are not included.

The main objective of this study is therefore the development of a new model system for dike breaching initiated by wave overtopping in order (i) to improve the capability of dike failure assessment, (ii) to estimate the warning time and (iii) to define initial conditions for flood propagation modelling (Section 1.1). The new model system should overcome most of the limitations of available models:

- **Type of structure:** sea dikes made of a sand core and a clay cover with grass;
- **Cause of breaching:** wave overtopping, overflow, a combination of both and water infiltration in the dike, no assumed initial breach;
- **Type of model:** deterministic and more process-oriented model;
- **Model uncertainties:** sensitivity analysis and reliability analysis.
2.5.2. Methodology and procedure

The methodology and procedure to be adopted in this study are summarised in Figure 2.8. In view of (i) the large variety and complexity of the hydrodynamic and morphodynamic processes involved, (ii) the considerable computational efforts required by a sophisticated process-oriented model, (iii) the detailed input data required by a latter, a tiered modelling approach is adopted. The modelling strategy includes a model system made of (i) a preliminary simplified model and (ii) a detailed more process-oriented model.

Both preliminary and detailed models are modular:

- **Input module (physical and numerical):**
  - Water levels and waves;
  - Dike geometry and material parameters;
  - Model parameters.
- **Hydrodynamic module** (breach hydraulics):
  - Free surface flow (wave overtopping, overflow or combined flow);
  - Infiltration.
- **Morphodynamic module** (breach morphology):
  - Erosion and sediment transport;
  - Soil failure (mass instability).
- **Output module** (numerical and graphical):
  - Breach width and depth;

![Figure 2.8: Detailed methodology and procedure](image-url)
Breaching outflow hydrograph;
Time of breaching.

**Preliminary model** which is based on simple formulae and assumptions is particularly needed in order (i) to explore and identify problems and most important issues to be improved in the development of the detailed model, (ii) to get more familiar with simulated processes and (iii) to identify through sensitivity analyses the most uncertain parameters and improve their prediction.

**Detailed model** is essentially based on preliminary model but: (i) some assumptions are removed and (ii) additional aspects of the breaching process are simulated. Among new aspects which are included, the following are worth to be mentioned: (i) breach initiation, (ii) water infiltration in the dike, (iii) grass and cover instability. Moreover a practical process-oriented tool for dike breaching simulation has to be achieved.

The study procedure follows six working steps:

1. **Preliminary hydrodynamic module**: only free surface flow is included. Wave overtopping is calculated with empirical formulae, combined flow and overflow with modified weir formulae and steady non-uniform free surface flow equations.

2. **Preliminary morphodynamic module**: available simple models, derived in case of overflow, are applied and eventually modified to account for waves. Breach initiation is rather simplified and the process starts in a weak section at the inner slope. Headcut advance is simulated as a continuous process and grass and cover sliding is neglected. Sand erosion is calculated with a 1D morphodynamic model and breach slopes instability is very simplified.

3. **Preliminary model uncertainties and validation**: sensitivity analysis of all input parameters, Monte Carlo and Latin Hypercube simulations are used to get a complete scenario of model uncertainties. Tentative validation is performed against data from experienced dike failures and from laboratory tests on homogeneous sand dikes with and without a clay cover.

4. **Detailed hydrodynamic module**: both free surface flow and infiltration are included. RANS-VOF Model (COBRAS) is used as an alternative to preliminary hydrodynamic module for free surface flow. Infiltration due to wave overtopping is calculated with simple models.

5. **Detailed morphodynamic module**: available simple models, derived in case of overflow, are still applied and eventually modified to account for waves, but additional processes are included. Breach initiation is estimated according to an initial scenario approach. Grass set-off and cover sliding and up-lift are calculated with limit equilibrium approach. Headcut advance in clay and sand-clay scour is simulated with discrete models. The width of the initial breach channel is also calculated and not imposed.

6. **Model uncertainties and validation**: sensitivity analysis of more uncertain input parameters and Latin Hypercube simulations are used to look at model uncertainties. Comparison with results from preliminary model and tentative validation are performed on the same data set used for the preliminary model.
3. Preliminary model: development, implementation and tentative validation

A preliminary model represents the first step of the tiered-approach proposed in Section 2.5 for dike breach modelling. The main objective is to provide an overview of the whole breaching process, as a starting base for further development. The model is based on simple formulae and assumptions, but includes most of the processes involved during dike breaching. The model is made of a set of modules (Figure 3.1). The most important are hydrodynamic (Figure 3.1a and Section 3.1.1) and morphodynamic (Figure 3.1b and Section 3.1.2) modules.

The hydrodynamic module includes wave overtopping, combined flow and overflow at the dike and through the breach.

The morphodynamic module includes grass, clay and sand erosion, as well as mass instability, according to a sequence of six breaching phases up to the final breach (Section 3.1.2). Model uncertainties are quite high and mainly related to input parameters, especially sea and material properties (Sections 3.1.1.4 and 3.1.2.4).

In this Chapter, the development, implementation and tentative validation of the preliminary model are addressed, but more details are provided by D’Eliso et al. (2006b):

- Mathematical formulation: description of simulated processes and uncertainties (Section 3.1);
- Model implementation: description of modules, discussion of results and tentative validation (Section 3.2);
- Overview of results and limitations of the model (Section 3.3).

In Figure 3.1, the shape of the boxes indicates different types of information: (i) *slightly rounded*: simulated process and approach, (ii) *rectangular*: selected equations with references and main hypotheses (bold font), and referring models, (iii) *rectangular without a corner*: hypotheses of the present model, (iv) *rounded shape*: new aspects.
3.1. Mathematical formulation

Past dike failures, as well as available laboratory and field tests, show that dike breaching is a complex 3D physical process, resulting from several mechanisms which occur simultaneously or in cascade (Chapter 2). A preliminary model should simulate only a very simplified process. General assumptions are:
• 2D + 2D process: cover erosion and failure is calculated with a fully 2D model, neglecting the width of the erosion hole, whilst in the core, breach width is also parametrically included;
• Water infiltration in the dike is completely neglected;
• Interaction between the clay cover and the sand core is neglected and only the erosion of homogeneous materials is included;
• Hydrodynamic and morphodynamic modules are not coupled.

3.1.1. Hydrodynamic module

Hydrodynamic module includes flow calculation along the dike profile and through the breach. Wave overtopping represents the main hydrodynamic load (Figure 2.1a), but also combined flow (Figure 2.1b1) and overflow (Figure 2.1b2) can be simulated. In addition, overflow equivalent to wave overtopping can be reproduced.

The hydrodynamic module uses the local wave climate at the dike toe as input. Wave propagation from deep water to the dike toe is not included, but can be calculated with wave propagation models, such as solvers of the elliptic (Berkhoff, 1972) or parabolic (Kirby, 1986) mild slope equation. Both waves (H, T) and mean water level (MWL) can be specified in two ways:

1) Sea parameters: significant wave height (Hs), peak period (Tp) and still water level (SWL);

   1. Waves (H(t), T(t))
   A theoretical wave energy spectrum is calculated from the sea parameters. JONSWAP spectrum (Goda, 1985; Hasselmann et al., 1973) and TMA spectrum (Hughes, 1984; Kitaigorodskii et al., 1975; Thompson & Vincent, 1983), explicitly derived for finite water depth, are included (Figure 3.2a). A time series of water surface (η) over the still water level (SWL) is generated from the wave energy spectrum (Figure 3.2b) making use of the Deterministic Spectral Amplitude (DSA) method (Hughes, 1984) and applying the Inverse Fast Fourier Transform (IFFT). A time series of wave height (H) and period (T) results from 0-Up Crossing analysis of the time series of the water surface (η).

   2. Mean water level (MWL(t))
   Meteorological storm surge (hs), tidal variation (ht) and wave-induced set-up (hset-up) are theoretically calculated and added to the still water level (SWL) to generate a time series of mean water level (MWL). Simplified steady storm surges (Dean & Dalrymple, 1993; Kamphuis, 2000), sinusoidal tides and an approximate value of 5%Hs of wave-induced set-up (USACE, 2002) are used.

2) Time series of wave height (H), period (T) and mean water level (MWL) at the dike toe.

   Selected models are used to calculate flow velocity (v), flow depth (h), discharge over the dike (q) or through the breach (Qb) and backwater level (hp).
3.1.1.1 Wave overtopping

Wave overtopping is a time-space dependent 3D process (Figure 3.3), including:
- Distribution of waves at the dike along the dike line (Figure 3.3a);
- Flow variation within each single overtopping event and interaction between two successive overtopping events (Figure 3.3b);

Figure 3.3: Wave overtopping at a real sea dike (see Annex C for references)
- Different shapes of the overtopping tongue (Figure 3.3c, d);
- Distribution of wave overtopping at each dike section.

Wave overtopping calculated in the model is a 2D process, assuming that:
- Wave distribution along the dike line is neglected;
- Each overtopping event is well defined and distinct from the preceding and the following;
- The shape of an overtopping tongue is not included, the overtopping discharge is given per unit width and only normal wave incidence is considered (2D model);
- Time series of waves is assumed to be a sequence of regular waves defined by their height (H) and period (T);
- Each wave cycle and corresponding overtopping event lasts its wave period (T). Within each event, the discharge (q) varies with time, while the Froude number (Fr) is kept constant (Figure 3.5).

Flow velocity (v) and depth (h) along the dike profile are calculated using the formulae proposed by Schüttrumpf & Oumeraci (2005) and are summarised in Figure 3.4. More details are available in Schüttrumpf & Oumeraci (2005).

Figure 3.4: Wave overtopping at sea dikes: definition sketch and formulae according to Schüttrumpf & Oumeraci (2005)
These formulae have been derived from laboratory tests over a smooth plane surface. In principle, they cannot be applied when the inner slope erodes and scour holes appear (Section 2.2.3). Nevertheless, the erosion model applied to the clay cover (Temple & Hanson, 1994; Temple & Moore, 1997) makes use of the flow over the initial inner slope ($\beta$) during the whole breaching process (Section 3.1.2.2).

The resulting flow discharge ($q$) obtained from the formulae in Figure 3.4, is averaged within a wave cycle. For erosion calculation, it is necessary to also include flow variation within the wave cycle, which takes a wave period ($T$). For this purpose, the following procedure is set up in the model:

- Each wave cycle is divided into a given number of time steps, greater or equal to three ($N_{pw} \geq 3$). A minimum value of five time steps per wave is suggested in order to give a detailed description of the flow;
- Peak overtopping discharge ($q_p$) is calculated by multiplying the averaged discharge ($q$) by a factor (const) in the tentative range of $2 \div 3$:

$$q_p = \text{const} \cdot q \quad [3.1]$$

- Wave overtopping discharge within the wave cycle ($q$) has a triangular shape (Figure 3.5a);
- The local Froude number ($Fr$), which is defined at each section along the dike profile, is kept constant within the wave cycle (Figure 3.5b). Local values of flow velocity ($v$) and depth ($h$) are used as characteristic velocity and flow length:

$$Fr = \frac{v}{\sqrt{gh}} \quad [3.2]$$

- Flow velocity ($v$) and depth ($h$) within the wave cycle are calculated at each dike section with eq. [3.3] (Figure 3.5c, d):

$$q = hv; \quad Fr = \frac{v}{\sqrt{gh}} = \text{const} \quad \Rightarrow \quad h = \left( \frac{q}{Fr \sqrt{g}} \right)^{2/3}; \quad v = \frac{q}{h} \quad [3.3]$$

Backwater level ($h_p$) is calculated as a function of volume of overtopping ($V_O$) as specified in Section 3.1.1.2. Volume of overtopping ($V_O$) is given for each wave by the area associated with the discharge $q$ (Figure 3.5a). An arbitrary width of the overtopping tongue ($B_O$) may be assumed:

$$V_O = \int_0^{B_O} \int_0^T q(y,t) \, dt \, dy \quad \Rightarrow \quad V_O = qB_O \frac{T}{2} \quad [3.4]$$
3.1.1.2 Overflow

Overflow includes (i) overflow head \( h_{\text{Over}} \) at the dike, which is the mean water level (MWL) over the dike height \( H_d \), i.e. \( h_{\text{Over}} = \text{MWL} - H_d \), and (ii) flow through the breach \( h \).

The overflow head \( h_{\text{Over}} \) due to high mean water level is a quasi-2D process. In principle, it can be steady or unsteady depending on the variation rate of the mean water level. Recorded data indicate that the process evolves over time slower than breach widening \( \frac{dH_b}{dt} \). In Figure 3.6, data recorded in front of Sylt Island, in the North Sea (Germany) and provided by Amt für Ländliche Räume (ALR) of Husum, show one of the highest mean water level variation rate in the last 50 years. Nevertheless, this variation reaches an average value of 0.07 mm/s and a maximum of 0.23 mm/s, while during storm surge of 1953 in the Netherlands, breach widening \( \frac{dH_b}{dt} \) reached 13.3 mm/s.

The flow through the breach \( h \) varies faster, but still slower than breach widening \( \frac{dH_b}{dt} \). Field experiments performed at the ZWIN Channel in 1994 (Visser et al., 1996), showed that in case of non-cohesive sea dike, with a 2.6 m height \( H_d \), with an initial averaged overflow head \( h_{\text{Over}} \) of 0.09 m, the dike crest started decreasing after 6.5 min and the flow become sub-critical in the breach channel after 23 min. Flow depth in the breach \( h \) grew in 16.5 min from zero up...
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to the order of magnitude of the initial dike height ($H_0$), resulting in variation rate of 2.63 mm/s, while breach widening ($dB_b/dt$) reached about 19 mm/s.

Figure 3.6: Mean water level variation at the dike (Amt für Ländliche Räume of Husum)

The overflow head ($h_{\text{Over}}$) and the flow through the breach ($h$) are calculated in the model as a 2D sequence of steady states, according to several available breach models (D’Eliso et al., 2005).

Flow velocity ($v$) and depth ($h$) are calculated solving the continuity and Bernoulli equations for steady ($\partial v/\partial t = 0$) non-uniform ($\partial v/\partial x \neq 0$) free surface flow:

a) Continuity equation:
   (i) Overflow discharge: $q = hv$ \hspace{1cm} [3.5]
   (ii) Discharge through the breach: $Q_b = B_b hv = \text{const}$ \hspace{1cm} [3.6]

b) Bernoulli equation (total flow energy): $\frac{dH}{dx} = -J; \hspace{1cm} H = z + h + \frac{v^2}{2g}$ \hspace{1cm} [3.7]

The energy slope ($J$) is calculated in the model with the Manning’s formula:

$$J = \frac{n^2 v^2}{R_b^{4/3}} \hspace{1cm} \text{with} \hspace{1cm} n = \text{Manning's roughness}$$ \hspace{1cm} [3.8]

Boundary conditions (BC) are required to solve eqs. [3.5]-[3.7] (Figure 3.7):

1) Transitional flow (Figure 2.5a, b, c and Figure 3.7a):
   - Downstream boundary condition for sub-critical flow at the outer slope and at the dike crest: overflow discharge ($q$) or discharge through the breach ($Q_b$) and critical flow depth ($h_{\text{cr}}$);
• Upstream boundary condition for super-critical flow at the inner slope: overflow discharge (q) or discharge through the breach (Q_b) and critical flow depth (h_{cr});

2) Non-transitional flow -sub-critical everywhere- (Figure 2.5d and Figure 3.7b):
• Downstream boundary condition for sub-critical flow at the inner slope: overflow discharge (q) or discharge through the breach (Q_b) and backwater level (h_p).

Finite difference numerical scheme (explicit forward calculation):

$$\frac{H_{i+1} - H_i}{\Delta x_{i+1}} = -J_{i+1} \Rightarrow z_{i+1} + h_{i+1} + \frac{v_{i+1}^2}{2g} = z_i + h_i + \left(1 - \alpha\right) \frac{v_i^2}{2g} - J_{i+1} \Delta x_{i+1}; \quad J_{i+1} = \frac{J_i + J_{i+1}}{2}$$

Correction for breach section contraction-expansion ($\alpha$):

$$\alpha = \left(1 - \frac{\Omega_{h_{i+1}}}{\Omega_{h_i}}\right) \frac{v_{i+1}^2}{2g} \quad \text{for} \quad \Omega_{h_{i+1}} \leq \Omega_{h_i}; \quad \alpha = \left(1 - \frac{\Omega_{h_{i+1}}}{\Omega_{h_i}}\right) \frac{v_{i+1}^2}{2g} \quad \text{for} \quad \Omega_{h_{i+1}} > \Omega_{h_i}$$

Discharge (weir formula of Poleni) and critical flow depth

I) Overflow head:
$$q = \frac{2}{3} \mu_{O\text{ver}} \left(\sqrt{2g}\right) h_{i+1}^{1.5}$$
$$h_{i} = \left(\frac{q}{g}\right)^{1/3}$$

II) Flow through the breach:
$$Q_b = \frac{2}{3} \mu_{O\text{ver}} \left(\sqrt{2g}\right) B_b (\text{MWL} - Z_b)^{1.5}; \quad h_{i} = \left(\frac{Q_i^2}{gB_i^2}\right)^{1/3}$$

Discharge coefficient ($\mu_{O\text{ver}}$) (Visser, 1998b):
- Breach initiation and formation phases (broad crested weir): $\mu_{O\text{ver}} \approx 0.58$
- Breach development -full breach- (contracted flow): $\mu_{O\text{ver}} \approx 0.75$

Discharge coefficient ($\mu_{O\text{ver}}$) may be kept constant at the contracted flow value.

Backwater level coefficient ($S_Q$) for correction of overflow discharge ($q_{o\text{ver}}$) and discharge through the breach ($Q_{b\text{ver}}$):

$$S_Q = \left(1 - \left(\frac{h_p - Z_s}{h - Z_s}\right)^{2.383}\right)^{0.383} \quad \text{for} \quad h_p > Z_s; \quad S_Q = 1 \quad \text{for} \quad h_p \leq Z_s \Rightarrow q_{o\text{ver}} = S_Q q \quad \text{and} \quad Q_{b\text{ver}} = S_Q Q_b$$

Figure 3.7: Overflow at sea dikes: definition sketch and equations used

Flow through the breach (h) becomes sub-critical everywhere when the total flow energy at the breach channel exit ($H_p$) is greater than total flow energy associated to the critical flow depth at each section of the breach channel ($H_{cr,max}$):

$$H_p \geq H_{cr,max} \quad \text{with} \quad H_p = h_p \quad [3.9]$$
Backwater level \( h_p \) is calculated assuming that the volume of water released behind the dike \( V_O \) is uniformly distributed in the polder area \( A_p \). It is function of the initial backwater level \( h_{p,0} \), if any, and of the volume of water per unit area \( V_O/A_p \):

\[
V_O = qB_0 \Delta t; \quad V_O = Q_b \Delta t \quad [3.10]
\]

\[
h_p = h_{p,0} + \frac{V_O}{A_p} \quad [3.11]
\]

**Equivalent overflow**, i.e. overflow discharge \( q_{eq} \) equivalent to wave overtopping discharge \( q \), may also be defined in the model as the overtopping discharge at the landward end of the dike crest, associated with a given significant wave height \( H_S \) and peak period \( T_p \). It is calculated by using the formulae in Figure 3.4. This is achieved by substituting \( H \) with \( H_S \), \( L_0 \) by \( L_{0,p} = gT_p^2/2\pi \) and using coefficients for irregular waves (Schüttrumpf & Oumeraci, 2005).

### 3.1.1.3 Combined wave overtopping and overflow

Combined wave overtopping and overflow, i.e. combined flow, is the sum of overtopping caused by waves and overflow caused by a high mean water level. When the overflow head \( h_{Over} \) or the flow through the breach \( h \) is much higher than the wave amplitude \( H/2 \), it reduces to overflow. From a physical point of view, it is still not definitely clear which influence waves has on combined flow, but combined flow discharge may result from the combination of two opposite effects: (i) the discharge increases due to the impulsive forces that waves transfer to the flow and (ii) the discharge decreases due to the increased dissipation at the free surface produced by waves. Which effect is prevailing on the other will mainly depend on the mutual influence of waves on one hand and overflow on the other hand.

Simulated combined flow is reduced in the models to a 2D sequence of steady states, similarly to overflow (Sections 3.1.1.1-3.1.1.2).

Flow velocity \( v \) and depth \( h \) are calculated solving steady non-uniform free surface flow equations, as in case of overflow (Section 3.1.1.2). The effects of waves on combined flow are all included in the discharge (Bleck et al., 2000; Oumeraci et al., 2001; Oumeraci et al., 1999). A combined flow discharge \( q_{Comb} \) or discharge through the breach \( Q_{b,Comb} \) is defined as a wave-averaged overflow discharge \( q \), calculated with a modified weir formula by assuming regular and sinusoidal waves (Figure 3.8).

The combined flow region is defined between a zero overflow head \( h_{Over} = 0 \) and an overflow head much higher than the wave amplitude \( h_{Over} >> H/2 \), defining two **flow transitions and two sub-regions** (Figure 3.8b).
Chapter 3 Preliminary model: development, implementation and tentative validation

**a) Combined flow: definition**

<table>
<thead>
<tr>
<th>Wave $\eta(t)$</th>
<th>Simulated combined flow</th>
<th>Real combined flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{MWL}$</td>
<td>$H_d$</td>
<td>$h_{Over}$</td>
</tr>
<tr>
<td>Overflow $H_d$</td>
<td>$h_{real}$</td>
<td></td>
</tr>
</tbody>
</table>

**b) Combined flow: sub-regions**

- $h_{Over} = 0$
- $h_{Over} = H_d/2$
- $h_{Over} >> H_d/2$

**Combined flow discharge ($q_{Comb}$):**

$$q_{Comb} = -\frac{2}{3} \mu_{Comb} \left( \sqrt{2g} \right) h_{Comb}^{1.5}$$

**Combined flow discharge through the breach ($Q_{b,Comb}$):**

$$Q_{b,Comb} = -\frac{2}{3} \mu_{Comb} \left( \sqrt{2g} \right) B_h h_{Comb}^{1.5}$$

**Discharge coefficient** (Oumeraci et al., 2001):

$$\mu_{Comb} = 0.4728 \xi_0; \quad \xi_0 = \frac{\tan \alpha}{\sqrt{H/L_0}}; \quad L_0 = \frac{gT^2}{2\pi}$$

**Overflow (overflow head):**

$$h_{Over} = h_{MWL} - H_d$$

**Wave (water surface):**

$$\eta(t) = \frac{H}{2} \sin \left( \frac{2\pi}{T} t \right)$$

**Combined flow (flow depth):**

$$q_{Comb} = \left[ \frac{1}{T} \int_0^T \left( \frac{2}{3} \mu_{Comb} \sqrt{2g} \left( h_{Over} + \eta(t) \right)^{1.5} \right) dt \right]^{1/2}$$

$$h_{Comb} = \left[ \frac{1}{T} \int_0^T \left( h_{Over} + \eta(t) \right)^{1.5} dt \right]^{2/3}$$

Figure 3.8: Combined flow at sea dikes: definition sketch and combined flow discharges according to Bleck et al. (2000)

Adjustments are necessary at both transitions (Figure 3.9):

**a) Transition 1: wave overtopping - combined flow ($h_{Over} = h_{MWL} - H_d = 0$)**

Wave overtopping discharge ($q$) should equal combined flow discharge ($q_{Comb}$) for zero overflow head ($h_{Over}$), but using selected models, wave overtopping discharge ($q$) is greater than combined flow discharge ($q_{Comb}$). In the first combined flow sub-region ($0 < h_{Over} \leq H_d/2$), a corrected combined flow discharge ($q_{Comb}$) is then approximately calculated (Figure 3.9):

$$q_{Comb} = \left| q_{Comb} - \left( \frac{H_d/2 - h_{Over}}{H_d/2} \right) (q_{Comb} - q) \right|_{h_{Over}=0}$$

**b) Transition 2: combined flow - overflow ($h_{Over} >> H_d/2$)**

Overflow discharge ($q$) should asymptotically equal combined flow discharge ($q_{Comb}$) for an overflow head ($h_{Over}$) much higher than the wave amplitude (H/2), when the influence of waves becomes negligible. Nevertheless, applying the selected models, the combined flow discharge ($q_{Comb}$) corresponding to an overflow head ($h_{Over}$) higher than the wave amplitude (H/2), is either lower or higher than the overflow discharge ($q$), depending on wave height (H) and period.
Breaching of sea dikes initiated by wave overtopping

C. D’Eliso

(T). In the second combined flow sub-region \((h_{\text{Over}} > H/2)\), a corrected combined flow discharge \(q_{\text{comb}}\) is provided (Figure 3.9):

\[
q_{\text{comb}} = \frac{2}{3} \sqrt{2g} \left( \mu_{\text{Comb}} h_{\text{Comb}}^{1.5} + \left( \frac{h_{\text{Over}} - H/2}{h_{\text{Comb}} - H/2} \right) \left( \mu_{\text{Comb}} dh^{1.5} - \mu_{\text{Comb}} h_{\text{Comb}}^{1.5} \right) \right) 
\]

Figure 3.9: Combined flow discharges for an incident wave with \(H_s = 3.0\) m, \(T_p = 10.0\) s (see also Figure 3.8)

Combined flow discharge through the breach after correction \(Q_{b,\text{comb}}\) is simply given by eq. [3.14]:

\[
Q_{b,\text{comb}} = B_s q_{\text{comb}} 
\]

The mutual influence of wave action and pure current flow can be estimated by the normalised difference \(\varepsilon\) between corrected combined flow \(q_{\text{comb}}\) and overflow discharge \(q\). Wave action may be considered negligible when this difference \(\varepsilon\) is less than about 2 \%. During breach formation (Figure 2.4), wave action becomes negligible. In fact, for the three waves of Table 3.1, the difference in the discharges \(\varepsilon\) is equal to 2 \% already for overflow head smaller than half dike height \((h_{\text{Over}} < H_d/2)\).

As a result of the selected wave overtopping (Figure 3.4), overflow (Figure 3.7) and combined flow (Figure 3.8 and Figure 3.9) models, a summary of
equations for flow discharge calculation during dike breaching is provided in Table 3.2.

Table 3.1: Mutual influence of wave action and pure current flow in combined flow for a dike of \( H_d = 7.00 \) m

<table>
<thead>
<tr>
<th>( H ) [m]</th>
<th>( T ) [s]</th>
<th>( \varepsilon = \frac{q_{\text{Comb}} - q_{\text{Over}}}{q_{\text{Comb}}} )</th>
<th>1%</th>
<th>2%</th>
<th>5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>6</td>
<td>Overflow head ( h_{\text{Over}} ) [m]</td>
<td>0.85</td>
<td>0.75</td>
<td>0.65</td>
</tr>
<tr>
<td>2.5</td>
<td>6</td>
<td>3.25</td>
<td>2.50</td>
<td>1.85</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>9</td>
<td>4.30</td>
<td>3.25</td>
<td>2.35</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2: Selected equations for flow discharge during breaching

<table>
<thead>
<tr>
<th>Overflow head</th>
<th>Flow</th>
<th>Discharge formula</th>
<th>Equation type</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_{\text{Over}} \leq 0 )</td>
<td>Overtopping</td>
<td>Figure 3.4</td>
<td>Continuity</td>
</tr>
<tr>
<td>( 0 \leq h_{\text{Over}} \leq H/2 )</td>
<td>Combined flow</td>
<td>eq. [3.12]</td>
<td>Broad crested weir</td>
</tr>
<tr>
<td>( H/2 \leq h_{\text{Over}} \leq h(\varepsilon=2%) )</td>
<td>Combined flow</td>
<td>eq. [3.13]</td>
<td>Broad crested weir</td>
</tr>
<tr>
<td>( h_{\text{Over}} \geq h(\varepsilon=2%) )</td>
<td>Overflow</td>
<td>Figure 3.7</td>
<td>Broad crested weir</td>
</tr>
<tr>
<td>( h_{\text{Over}} \geq h(\varepsilon=2%) )</td>
<td>Overflow (full breach)</td>
<td>Figure 3.7</td>
<td>Contracted flow</td>
</tr>
</tbody>
</table>

3.1.1.4 Uncertainties related to hydrodynamic aspects

The hydrodynamic module is based on empirical formulae and simple equations. Therefore only few and simple input parameters are needed:

- **Sea parameters**: wave height \( H \), wave period \( T \), mean water level (MWL);
- **Dike geometry**: dike height \( H_d \), crest width \( B_d \), outer \( 1:m \) and inner \( 1:n \) slopes.

The most uncertain and important parameters are sea parameters. Only direct monitoring of nearshore hydrodynamics may reduce them.

Model parameters include coefficients calibrated on laboratory tests, such as the friction coefficient \( f \) in Figure 3.4 and the discharge coefficients of combined flow \( \mu_{\text{Comb}} \) and overflow \( \mu_{\text{Over}} \) discharges (Figure 3.7 and Figure 3.8). Although they may depend on the data set used for the model derivation, they are much less uncertain than input parameters.

3.1.2. Morphodynamic module

The morphodynamic module includes the calculation of breach formation and growth from the initiation to the final breach, addressing grass, clay and sand erosion and instability.

Dike breaching is a time-space dependent 3D process, but in the model it is reduced to a 2D + 2D process. All selected models are 2D, but making some
assumptions, the 3D shape of the breach may also be described. The model reproduces the whole breaching process in six phases: (i) three for erosion, instability and failure of the clay cover with grass (Figure 3.10a) and (ii) three for erosion, instability and failure of the dike core (Figure 3.10b):

1. Phase 1: Failure of grass due to gradual erosion;
2. Phase 2: Local erosion of the clay cover;
3. Phase 3:
   - Phase 3a: Scour erosion and headcut advance in the clay cover up to sand core exposure to flow action;
   - Phase 3b: Instantaneous sliding and failure of the clay cover, initial breach channel at the inner slope with bed of sand and slopes of clay;
4. Phase 4: Dike crest shortening due to scour erosion in sand, progressive failure of the clay cover at the crest;
5. Phase 5:
   - Phase 5a: Dike crest lowering and breach widening due to scour erosion in sand and breach slopes instability, progressive failure of clay cover at the outer slope, driven by wave overtopping (t < t₀, Figure 2.4);
   - Phase 5b: Dike crest lowering and breach widening due to scour erosion in sand and breach slopes instability, progressive failure of clay cover at the outer slope, driven by combined flow (t > t₀, Figure 2.4);
6. Phase 6:
   - Phase 6a: Full breach, breach widening due to scour erosion in sand and breach slopes instability driven by super-critical flow in the breach (transitional flow);
   - Phase 6b: Breach widening due to scour erosion in sand and breach slopes instability up to equilibrium final breach driven by sub-critical flow in the breach (non-transitional flow).

General assumptions of the simulated processes are:

- Breach initiation (phase 1) is detected assuming weak sections along the inner slope, where grass has a lower protection capability against erosion;
- Erosion is calculated only at the inner slope, where it is much higher than at the outer slope and dike crest (D'Eliso et al., 2005);
- Erosion of the clay cover (phases 2 and 3) is only calculated at sections where grass has been eroded;
- Headcut advance in clay (phase 3a) is calculated using a continuous averaged approach (Section 2.2.3);
- As soon as the sand core is exposed to the flow (phase 3b), the clay cover is instantaneously removed and the headcut erosion in sand-clay scour is neglected;
- Breach slopes instability is calculated using a simplified discrete approach;
- Breach shape is rectangular in phases 4, 5, 6a (Hassan, 2002; Rozov & Chanson, 2005) and trapezoidal at the end of phase 6b (Coleman et al., 2002).
Chapter 3 Preliminary model: development, implementation and tentative validation

Loading case 1: wave overtopping
Main assumption: each wave of the time series is assumed to be a sinusoidal wave with a certain height ($H$) and period ($T$)

Loading case 2: combined wave overtopping and overflow
Main assumption: effects of waves on combined flow are concentrated in the combined flow discharge ($q_{comb}$)

Simulated process: erosion of grass
Erosion: cumulated excess shear stress, grass erosion up to failure is due to repetition of flow action
Leading parameter: Total Manning's roughness ($n_{tot}$)

Simulated process: local clay erosion is small rills and holes
Erosion: cumulated excess shear stress
Leading parameter: erodibility coefficient ($k_d$)

Simulated process: scour erosion and headcut advance
1. Erosion: cumulated excess shear stress, shear stress at the headcut base
Leading parameter: erodibility coefficient ($k_d$)
2. Headcut advance: simplified continuous averaged approach, definition of an average headcut advance rate
Leading parameter: headcut erodibility coefficient ($K_h$)

Simulated process: erosion of sand core and breach formation
Flow: transition between wave overtopping and combined flow and between combined flow and overflow

Breach morphology: vertical slopes (rectangular shape)
Erosion: sediment transport capacity and sediment mass conservation (vertical and lateral erosion)
Mass instability: simplified instability analysis
Leading parameters: energy slope ($J$), sediment transport discharge ($Q_s$)

Simulated process (phases 6): breach widening up to the equilibrium final breach
Flow: transitional and non-transitional flow
Breach morphology: final non vertical slopes (trapezoidal shape)
Erosion and mass instability as in Phase 4 and 5.

Figure 3.10: Breaching of sea dikes initiated by wave overtopping: modelling phases and simulated processes
The morphodynamic module uses the outcomes of the hydrodynamic module (Section 3.1.1) and grass, clay and sand properties as inputs. The selected models calculate the time of breaching \((t_b)\), the headcut height \((H_{hi})\), the breach height \((H_b)\) and the breach width \((B_b)\).

### 3.1.2.1 Erosion of the grass cover

The grass cover fails due to gradual erosion, mostly as a particle detachment due to flow-induced shear stress on clay soil (phase 1 of Figure 3.10). Failure starts at a randomly located section at the inner slope.

The time of grass failure \((t_{gf})\) and the erosion depth at grass failure \((\Delta z_g)\) are calculated using permissible tractive force approach (Section 2.2.2). The selected model (Temple & Hanson, 1994), derived from field tests on earth spillways and embankments with grass, is based on the effective bottom shear stress \((\tau_{0,e})\):

\[
\tau_{0,e} = \tau_0 \left(1 - C_f \right) \left( \frac{n_c}{n_{tot}} \right)^2 \quad \text{with} \quad \tau_0 = \rho_w g h J
\]

Flow action is included through the bottom shear stress \((\tau_0)\) with flow depth \((h)\) and energy slope \((J)\). Influence of grass on the erosion is concentrated in the grass cover factor \((C_f)\) and in the ratio between Manning’s roughness of clay \((n_c)\) and total Manning’s roughness \((n_{tot})\).

The time of grass failure \((t_{gf})\) depends on the cumulated flow action, which is in the model, the integral over time of the effective bottom shear stress \((\tau_{0,e})\) and is defined when the integral is higher than a threshold value, function of the plasticity index of clay \((I_p)\):

\[
\int_0^{t_{gf}} \tau_{0,e} \, dt = 3600 \left( 9I_p + 50 \right) \quad \Rightarrow \quad t_{gf}
\]

Eq. [3.16] is derived under the assumption that the critical shear stress \((\tau_{0,cr})\) is much lower than the effective bottom shear stress \((\tau_{0,e})\) and negligible. It has to be noticed that in case of wave overtopping this assumption may be violated (Section 3.1.2.2).

The erosion depth at which the grass function fails \((\Delta z_g)\), is conventionally assumed equal to a percentage of the stem length of the grass \((L_s)\). A value of 90% of \(L_s\) is suggested:

\[
\Delta z_g = 0.9L_s
\]
Energy slope \((J)\) is calculated in two ways, depending on the loading case considered:

- Wave overtopping: it is directly taken as the gradient of the total flow energy (eq. [3.7]), because both flow velocity \((v)\) and depth \((h)\) are known from the formulae in Figure 3.4;
- Combined flow: available models suggest a constant energy slope equal to the inner slope (NRCS, 1997). However in the model, it is calculated with eq. [3.8].

Manning’s roughness of clay \((n_c)\) is calculated as a function of the representative sediment size \(D_{75,c}\) (Lane, 1955):

\[
n_c = \frac{D_{75,c}^{1/6}}{14.23} \tag{3.18}
\]

Total Manning’s roughness \((n_{tot})\) includes the effects of both clay soil particles and grass. Available formulae strictly apply to sub-critical flows. Application of these formulae to super-critical small flow depths, as in case of wave overtopping at the inner slope, often yields unrealistic values:

- USDA’s formula (Temple et al., 1987): derived from field tests on sub-critical flows as a function of flow discharge \((q)\) and type of grass, through the curve retardance factor \((C_1)\). It provides very high unrealistic results for small flow depths, e.g. total Manning’s roughness of about 1.5 for a discharge \((q)\) of 0.005 \(m^3/sm\) and a curve retardance factor \((C_1)\) of 6 and it is therefore not given here, but provided in D’Eliso et al. (2006b);
- Samani & Kouwen’s formula (Kouwen & Li, 1980; Samani & Kouwen, 2002): derived from laboratory tests on sub-critical flows and several types of grass as a function of flow, type and grass conditions. The formula is not defined for flow depths \((h)\) lower than about 0.05 m and it is therefore not given here, but provided in D’Eliso et al. (2006b);
- Diàz’s formulae (García Diàz, 2005): derived from laboratory tests on small sub-critical flow depths and several types of grass as a function of the local Froude number \(Fr\) (eq. [3.2]), irrespective of the type of grass. The formulae are very simple and provide realistic results:

\[
n_{tot} = 0.0682Fr^{-0.9579} \quad \text{for} \quad \beta \leq 20^\circ \tag{3.19}
\]

\[
n_{tot} = 0.0994Fr^{-1.0085} \quad \text{for} \quad \beta > 20^\circ \tag{3.20}
\]

Extension to super-critical flow depths shows that for local Froude numbers \((Fr)\) higher than about 3, the influence of the inner slope angle \((\beta)\) becomes irrelevant and total Manning’s roughness \((n_{tot})\) asymptotically decreases to zero (Figure 3.11).
Summarising about the analysis of available formulae for the calculation of the total Manning’s roughness ($n_{\text{tot}}$):

- USDA’s and Samani & Kouwen’s formulae are not suitable in case of wave overtopping;
- Diàz formulae can tentatively be applied in case of overtopping and combined flow. In fact, although it is originally tested on sub-critical flows, (i) it is derived for small flow depths, (ii) it doesn’t include grass parameters which are affected by strong uncertainties and (iii) the flow action is included in the local Froude number ($F_r$), which contains information about the flow regime;
- There is no experimentally verified formula for the calculation of the total Manning’s roughness ($n_{\text{tot}}$) in case of wave overtopping, so that the three suggested approaches can be optionally selected in the model, but the use of Diàz’s formulae is recommended.

The grass cover factor ($C_t$) has experimentally been estimated as a function of the type of grass and provided in Temple et al. (1987). If the type of grass is unknown or an idealized scenario is simulated, values reported in Table 3.3 can be used in the model (Temple & Hanson, 1994).

<table>
<thead>
<tr>
<th>Type of grass</th>
<th>Grass cover factor $C_t$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good</td>
<td>0.75</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.50</td>
</tr>
<tr>
<td>Poor</td>
<td>0.25</td>
</tr>
</tbody>
</table>
Location of the incipient grass failure and breach initiation in the model is based on the definition of sections along the inner slope that are mostly exposed to erosion. Values of the grass cover factor ($C_f$) indicate in the model which dike section is most prone to fail. The grass cover factor ($C_f$) is normally distributed and randomly assigned at the inner slope by extractions from its probability distribution. Dike breaching starts where it assumes the lowest value. This approach is rather pragmatic than process-oriented, but based on the evidence that the initial breach is located at a “weak section” along the inner slope and suggests the development of an initial scenario approach for breach initiation (Section 4.1.2.1).

3.1.2.2 Erosion of the clay cover

After grass erosion and failure, clay cover erosion and instability develop in two successive phases and associated times:

a) Local clay erosion (phase 2 of Figure 3.10): key parameters of the selected model to calculate the vertical erosion depth ($dz$) are the headcut erodibility coefficient ($k_d$) and the critical shear stress ($\tau_{0,cr}$);

b1) Headcut erosion (phase 3a of Figure 3.10): calculation of the vertical erosion depth ($dz$) and the headcut advance ($dX$). Important parameters are the initial headcut height ($H_{H,0}$) and the headcut erodibility coefficient ($K_h$);

b2) Clay cover failure (phase 3b of Figure 3.10): initial breach channel height ($H_{b,0}$) and width ($B_{b,0}$).

a) Local clay erosion (phase 2)

The flow at the inner slope first concentrates in rills and small rivulets that deepen with time (Section 2.2.3) due to local clay erosion (phase 2 of Figure 3.10). Particle detachment mechanisms still dominate.

The vertical erosion depth ($dz$) in the model is proportional to the excess effective bottom shear stress ($\tau_{0,e}$-$\tau_{0,cr}$), using Duboy’s sediment transport equation, modified for rill erosion (Meyer, 1964):

$$\frac{dz}{dt} = k_d \left( \tau_{0,e} - \tau_{0,cr} \right)^a \Rightarrow dz = k_d \left( \tau_{0,e} - \tau_{0,cr} \right)^a dt \quad [3.21]$$

Where the effective bottom shear stress ($\tau_{0,e}$) is function of the flow depth ($h$) and the erosion depth at the previous time step ($dz_{(t-dt)}$):

$$\tau_{0,e} = \rho_w g \left( h + dz_{(t-dt)} \right) J \quad [3.22]$$
The parameter a is an erosion rate coefficient that should be estimated from field or laboratory data. Samani & Kouwen (2002) found a value of $a = 1.15$, but it is normally assumed equal to 1 (Temple & Hanson, 1994).

The erodibility coefficient ($k_d$) depends on clay properties and is calculated with the USDA's formula (Temple & Hanson, 1994). According to results by Samani & Kouwen (2002), the USDA’s formula doesn’t include the sediment size, but only the dry clay soil density ($\rho_{c,d}$) and the weight percentage of clay in the cohesive soil ($c_w$) (Figure 3.12):

$$k_d = 10^{-6} \frac{10\rho_w}{\rho_{c,d}} \exp \left[ -0.121c_w^{0.406} \left( \frac{\rho_{c,d}}{\rho_w} \right)^{3.10} \right]$$

[3.23]

Figure 3.12: Erodibility coefficient ($k_d$) as a function of clay soil properties

The critical shear stress ($\tau_{0,cr}$) is neglected in most of available models (D'Eliso et al., 2005). However, in the present model, it represents an important parameter. The critical shear stress ($\tau_{0,cr}$) is in the model proportional to mass concentration of particles (Osman & Thorne, 1988):

$$\tau_{0,cr} = 5.43 \cdot 10^{-6} \left( \rho_{c,d} \frac{\rho_{cw} - \rho_w}{\rho_{c,d} - \rho_w} \right)^{2.28}$$

[3.24]

Density of water-clay sediment mixture ($\rho_{cw}$) is equal to about 1100 kg/m$^3$.
The calculation of the erosion rate \( (dz/dt) \) by either neglecting or including the critical shear stress \( (\tau_{0,cr}) \), indicates that for small flow depths \( (h < 0.02 \text{ m}) \), the critical shear stress \( (\tau_{0,cr}) \) has to be included in the erosion calculation (Table 3.4).

Table 3.4: Influence of critical shear stress \( (\tau_{cr,0}) \) on local clay erosion rate \( (dz/dt) \)

<table>
<thead>
<tr>
<th>Water depth ( h ) [m]</th>
<th>Effective shear stress ( \tau_{0,e} ) [N/m²]</th>
<th>Erosion rate with critical stress ( dz/dt ) [mm/s]</th>
<th>Erosion rate without critical stress ( dz/dt ) [mm/s]</th>
<th>Error ( \varepsilon ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.005</td>
<td>17</td>
<td>0.0094</td>
<td>0.0099</td>
<td>5.3</td>
</tr>
<tr>
<td>0.05</td>
<td>33</td>
<td>0.0193</td>
<td>0.0198</td>
<td>2.6</td>
</tr>
<tr>
<td>0.1</td>
<td>332</td>
<td>0.1980</td>
<td>0.1985</td>
<td>0.3</td>
</tr>
<tr>
<td>0.2</td>
<td>663</td>
<td>0.3965</td>
<td>0.3970</td>
<td>0.1</td>
</tr>
<tr>
<td>0.3</td>
<td>995</td>
<td>0.5949</td>
<td>0.5954</td>
<td>0.1</td>
</tr>
<tr>
<td>0.4</td>
<td>1327</td>
<td>0.7934</td>
<td>0.7939</td>
<td>0.1</td>
</tr>
</tbody>
</table>

\( \varepsilon \) = 100 \times \frac{(dz/dt)_{cr} - (dz/dt)_{non-cr}}{(dz/dt)_{cr}}

Weight percentage of clay in the cohesive soil \( c_{wm} = 30 \% \)
Dry soil density \( \rho_{cd} = 1700 \text{ kg/m}^3 \)
Energy slope \( J = 0.33 \)

b) Headcut erosion and clay cover failure (phase 3)

Rills erosion concentrates at some dike sections forming a vertical headcut (Section 2.2.3) and the combination of scour erosion and headcut advance result in the clay cover erosion and failure (phase 3a of Figure 3.10).

Headcut initiation takes place when a vertical headcut starts migrating seaward (Section 2.2.3). The time of headcut initiation \( (t_H) \) is conventionally defined as the time corresponding to a vertical erosion depth \( (dz) \) in the clay cover greater than a threshold value, which is the initial headcut height \( (H_{H,0}) \) in the headcut erosion model. There are two possibilities in the model to specify the initial headcut height \( (H_{H,0}) \):

- As an input parameter, depending on the flow discharge \( (q) \), inner slope \( (1:n) \) and material properties;
- Equal to the critical flow depth \( h_{cr} \) (Temple & Moore, 1997):

\[
H_{H,0} = h_{cr} \tag{3.25}
\]

The second approach is preferred, but in the presence of waves, it is not directly applicable because a representative value of the flow discharge \( (q) \) is not univocally defined. The critical flow depth \( (h_{cr}) \) associated with the average discharge \( (q_{m}) \) may be used when only the number of waves \( (N_w) \) that result in a non-zero overtopping or combined flow discharge \( (q_i) \) are considered:

\[
q_m = \frac{1}{N_w} \sum_{i=1}^{N_w} q_i \tag{3.26}
\]
The vertical erosion depth \((dz)\) at the headcut base is calculated in the model with eq. [3.21], as in phase 2 of Figure 3.10, but the effective bottom shear stress \((\tau_{0,e})\) includes the effect of the impinging jet at the headcut base (NRCS, 1997; Robinson, 1992):

\[
\tau_{0,e1} = \rho_w ghJ \quad [3.27]
\]

\[
\tau_{0,e2} = 0.01 \rho_w gh \left( \frac{H}{H_{H,0}} \right)^{0.582} \quad [3.28]
\]

\[
\tau_{0,e} = \max(\tau_{0,e1}, \tau_{0,e2}) \quad [3.29]
\]

The headcut advance \((dX)\) is calculated by applying a continuous averaged empirical model, neglecting the explicit calculation of the cyclical mass instability from the vertical headcut. The selected model includes the influence of headcut height \((H_{H})\), flow discharge \((q)\) and material properties and defines a threshold for the headcut advance (Temple & Moore, 1997), as in Figure 3.13a (fixed headcut height \(H_{H} = 0.4 \text{ m}\)) and in Figure 3.13b (fixed flow discharge \(q = 0.02 \text{ m}^3/\text{sm}\)):

\[
dX = \begin{cases} \frac{1}{3600} C (A - A_0) dt & \text{for } A - A_0 > 0 \\ 0 & \text{for } A - A_0 \leq 0 \end{cases} \quad [3.30]
\]

Driving force \((A)\) is function of headcut height \((H_{H})\) and flow discharge \((q)\):

\[
A = \left( q H_{H} \right)^{1/3} \quad [3.31]
\]

The resisting force \((A_0)\) and the coefficient of headcut advance \((C)\) depend on material properties through the headcut erodibility coefficient \((K_h)\):

\[
A_0 = \begin{cases} 0.3048 \left[ 189 K_h^{0.5} \exp \left( \frac{-3.23}{\ln(101 K_h)} \right) \right]^{1/3} & \text{for } K_h > 0.01 \\ 0 & \text{for } K_h \leq 0.01 \end{cases} \quad [3.32]
\]

\[
C = \begin{cases} -0.79 \ln(K_h) + 3.04 & \text{for } K_h < 18.2 \\ 0.75 & \text{for } K_h \geq 18.2 \end{cases} \quad [3.33]
\]
The headcut erodibility coefficient \( (K_h) \) is a function of soil resistance and structure through four coefficients (NRCS, 2001):

\[
K_h = M_s K_b K_d J_s \tag{3.34}
\]

With:
- **Material strength number** \( (M_s) \): it represents the strength of an intact sample of soil, without variation within the mass and is a function of the unconfined compressive strength (UCS) of the soil. When specific information are lacking, classification of soil and relative unconfined compressive strength (UCS) are available (D’Eliso et al., 2006b; NRCS, 2001).
- **Particle size number of soil** \( (K_b) \): it includes the effect of the mean size of an intact block of material in the mass. In absence of specific information, the worse scenario is assumed: not cemented soil and particle size number of soil \( (K_b) \) equal to 1.
- **Inter-particle bond shear strength of soil** \( (K_d) \): it represents the shear strength along discontinuities in the soil mass and is a function of the liquid limit \( (w_l) \) and the weight percentage of clay in the cohesive soil \( (c\%)\).
- **Relative ground structure number** \( (J_s) \): it includes the effect of the soil structure and the mutual orientation of blocks and flow on headcut advance.

More details can be found in the original publication (NRCS, 2001).

**Time of cover erosion** \( (t_{ce}) \) and **failure** \( (t_{cf}) \) is defined when the scour hole at the headcut base exposes the sand core directly to flow action and when clay cover instantaneously collapses (Figure 3.14a), resulting in an initial breach channel with a rectangular cross-section (Figure 3.14b, **phase 3b** of Figure 3.10).
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The initial breach channel height \((H_{b,0})\) is equal to the cover layer thickness \((S_C)\). The initial breach channel width \((B_{b,0})\) is assumed as a function of the initial breach channel height (MacDonald & Landridge-Monopolis, 1984):

\[
B_{b,0} = (1 \div 3) H_{b,0} = (1 \div 3) S_c
\]

[3.35]

The ratio between the initial breach channel width \((B_{b,0})\) and the initial breach channel height \((H_{b,0})\) is given as non-dimensional input parameter.

### 3.1.2.3 Erosion of the non-cohesive core

Following the erosion and failure of the clay cover with grass, the erosion and instability of the sand core proceed due to flow-induced shear stress at the breach bottom and slopes, resulting in the wash-out of sand particles and breach slopes instability (Section 2.2.4). Vertical erosion is dominant at the beginning of the process (phase 4 of Figure 3.10), breach slopes instability becomes more relevant as breach deepens (phase 5a and 5b of Figure 3.10) and rapidly widens (phase 6a of Figure 3.10). Both erosion and instability becomes negligible close to the final breach (phase 6b of Figure 3.10), see Figure 3.10.

Mechanisms and governing equations are the same in all phases. Selected model is a 1D morphodynamic model, together with physically-based assumptions on breach morphology. Erosion and time associated with breaching are calculated according to four steps:

- Sediment transport discharge \((Q_{st})\);
- Eroded breach cross-section area \((dA_b)\);
- Vertical \((dz)\) and lateral \((db)\) erosion depths and cross-section updating;
- Breach slopes instability \((\text{FOS} < 1)\) and eventually, cross-section updating.

#### Sediment transport discharge \((Q_{st})\)

Through the breach depends on (i) non-uniform sediment transport regime, (ii) very high sediment concentration (iii) bed and suspended load and (iv) super-critical flow in the breach, but none of available models apply to these flow and sediment regimes (Section 2.3). Key
parameters are (i) the critical shear stress ($\tau_{0,cr}$) or the critical Shields’ parameter $\theta_{cr}$ (Shields, 1936) for bed discharge ($q_{sb}$) and (ii) settling velocity ($w_s$) (Ahrens, 2000; NBS, 1975; Soulsby, 1997; Van Rijn, 1993; Visser, 1998b) for suspended discharge ($q_{ss}$). The total sediment transport discharge ($Q_{st}$) is the sum of bed ($q_{sb}$) and suspended ($q_{ss}$) discharges, concentrated over the width at the breach bottom ($B_b$):

$$Q_{st} = B_b q_{st} \quad \text{with} \quad q_{st} = q_{sb} + q_{ss} \quad [3.36]$$

A selection of available sediment transport formulae is included in the model:

- **Bagnold-Visser’s formula** (Visser, 1988): based on Bagnold’s formula (Bagnold, 1966), derived according to an energetic approach. Total discharge ($q_{st}$) is a function of energy spent by the flow in transporting sediments. It has been already applied to breaching of sand dikes initiated by overflow (Visser, 1998b).

- **Bagnold-Bailard’s formula** (Bailard, 1981): based on Bagnold’s formula (Bagnold, 1966) and similar to Bagnold-Visser’s formula, but generally yields a much higher erosion rate (Visser, 1998b).

- **Smart’s formula** (Smart, 1984): modified Meyer-Peter & Muller’s formula (Meyer-Peter & Müller, 1948) in order to account for steep slopes (up to $\tan \beta = 0.2$) and derived from laboratory data for bed discharge ($q_{sb}$), but commonly used for total discharge ($q_{st}$). It has been already applied to breaching of dams initiated by overflow, providing good erosion estimates (Tingsanchali & Hoai, 1993), but predicts null sediment transport over a plane horizontal bed ($q_{st} \Rightarrow 0$ if $\beta \Rightarrow 0$). Therefore, when inner slope angle ($\beta$) is very small, the erosion is strongly underestimated.

- **Yang’s formula** (Yang & Molinas, 1982; Yang & Song, 1979): based on the hypothesis that the total discharge ($q_{st}$) is function of the unit stream power of the flow ($v\cdot J$) given by the product of the flow velocity ($v$) by the energy slope ($J$), and derived from laboratory data. It allows calculating sediment concentration at the bed and does not directly include inner slope angle ($\beta$). It has been applied to breaching of dams initiated by overflow (Hassan, 2002).

Concluding about the sediment transport formulae that have been selected, the following is noteworthy:

- Smart’s formula and further formulae that provide null sediment transport over a plane horizontal bed ($\beta = 0$), e.g. (Rickenmann, 1991) cannot properly simulate breach widening if calculation of lateral erosion (db) is explicitly included in the model;

- Bagnold-Bailard’s formula has no limitations on the bed discharge ($q_{sb}$). In fact, for an inner slope angle ($\beta$) close to the friction angle of sand ($\phi_s$), it predicts an infinite sediment transport ($q_{st} \Rightarrow \infty$ if $\beta \Rightarrow \phi_s$). Such limitation on the bed discharge ($q_{sb}$) has been introduced in the Bagnold-Visser’s formula (Visser, 1988);
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- Bagnold-Visser’s and Yang’s formulae describe the total discharge \( q_{st} \), explicitly include the effects of suspended sediments and are therefore recommended for use in the model (Table 3.5).

Table 3.5: Sediment transport formulae recommended in the model

<table>
<thead>
<tr>
<th>Formula</th>
<th>Equation</th>
</tr>
</thead>
</table>
| **Bagnold-Visser** (Visser, 1988) |Bed discharge (\( q_{sb} \)): \[
\frac{\tau_0 V}{\rho_w g \Delta (\tan \phi_s - \tan \beta) \cos \beta} \leq \zeta_2 (1 - \rho_s) D_{50,s} \gamma; \quad \zeta_2 = 2; \quad \epsilon_b = 0.13
\]

Suspended discharge (\( q_{ss} \)): \[
\frac{\tau_0 V}{\rho_w g \Delta w_s / \sqrt{\gamma (\cos \beta)}}; \quad \epsilon_s = 0.01
\]

Upper limitation of sediment concentration (\( C_s \)) and total discharge (\( q_{st} \)): \[
C_s = \frac{q_{st}}{q + q_{st}} \leq 0.6 \quad \Rightarrow \quad q_{st} \leq 1.5 q
\]

| **Yang** (Yang & Molinas, 1982; Yang & Song, 1979) | Total discharge (\( q_{st} \)): \[
Q_{st} = 10^{-3} \cdot 10^{C_s} \frac{q}{Q}
\]

Sediment concentration in mg/l (\( C_s \)): \[
\log_{10} C_s = 5.165 - 0.153 \log_{10}\left( \frac{w_s D_{50,s}}{\gamma} \right) - 0.297 \log_{10}\left( \frac{\nu_s}{w_s} \right) + 
\frac{1.780 - 0.360 \log_{10}\left( \frac{w_s D_{50,s}}{\gamma} \right) - 0.480 \log_{10}\left( \frac{\nu_s}{w_s} \right)}{\log_{10}\left( \frac{\nu_s}{w_s} \right)}; \quad \nu_s = \sqrt{g R_s}
\]

Relative density of sand particles (\( \Delta \)): \[
\Delta = \frac{\rho_{g,s}}{\rho_w} - 1; \quad \rho_{g,s} = 2700 \text{ kg/m}^3
\]

**Critical Shields’ parameter** (\( \theta_c \)) is a model input, assumed equal to 0.047 (Shields, 1936).

**Sediment fall velocity** (\( w_s \)) is optionally calculated in the model with two sets of formulae:

- **Van Rijn’s formula** (NBS, 1975; Van Rijn, 1993): already used in some available breach models (Hassan, 2002; Visser, 1998a), but not derived for the marine environment and therefore not given here;

- **Ahrens’s formula** (Ahrens, 2000): more recent, explicitly derived for coastal areas and simpler than the van Rijn’s formula. It is recommended for use in the model:

\[
w_s = C_1 \frac{\Delta g D_{50,s}^2}{\gamma} + C_2 \sqrt{\Delta g D_{50,s}} \quad [3.37]
\]

\[
C_1 = 0.055 \tanh(12A^{-0.59} \exp(-0.0004A)) \quad [3.38]
\]

\[
C_2 = 1.06 \tanh(0.016A^{-0.50} \exp(-120/A)) \quad [3.39]
\]

\[
A = \frac{\Delta g D_{50,s}^2}{\gamma^2} \quad [3.40]
\]
The kinematic viscosity of water ($\nu$) is calculated as a function of the water temperature $T$ (Ahrens, 2000):

$$\nu = 10^{-4}\left(c_0 + c_1 T + c_2 T^2\right)$$

$$c_0 = 1.82 \cdot 10^{-2}; \quad c_1 = -5.29 \cdot 10^{-4}; \quad c_2 = 6.90 \cdot 10^{-6}$$

Non-equilibrium sediment transport discharge can be included in the model using the Galappatti approach (Galappatti, 1983). Nevertheless, non-equilibrium sediment transport is limited to the initiation of sand erosion, which is relatively fast compared to the entire process of dike core erosion and final breach development (Visser, 1998b). Therefore, it might be neglected (Hassan, 2002).

Eroded breach cross-section area ($dA_b$) is calculated applying 1D sediment continuity equation (Exner eq.), derived under the assumptions that (i) variation of bed elevation is uniform over the cross-section and (ii) sediment transport is mainly unidirectional, along $x$ (Table 3.6):

$$\frac{\partial Q_{st}}{\partial x} + \left(1 - p_s\right) \frac{\partial A_b}{\partial t} = 0$$

Table 3.6: Solution of the Exner equation

<table>
<thead>
<tr>
<th>Finite difference numerical scheme (explicit forward calculation):</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{1}{2}\left(\frac{Q_{st,i+1} - Q_{st,i}}{\Delta x_{i+1,i}} + \frac{Q_{st,i} - Q_{st,i-1}}{\Delta x_{i,i-1}}\right) + (1 - p_s) \frac{A_{b,t,i+1} - A_{b,t,i}}{\Delta t} = 0 \Rightarrow A_{b,t,i+1}$</td>
</tr>
</tbody>
</table>

Vertical ($dz$) and lateral ($db$) erosion are concentrated close to the breach bottom where they have the same order of magnitude. Lateral erosion ($db$) decreases upward from the breach bottom to the free surface of the flow (Hassan et al., 1999). Sand core erosion assumes two shapes depending on the flow in the breach:

- **Transitional flow from super to sub-critical**: very fast erosion with almost vertical breach slopes (phases 4, 5 and 6a of Figure 3.10);
- **Sub-critical flow**: very low erosion with non-vertical breach slopes assumed at the equilibrium final breach (phases 6b of Figure 3.10).

Erosion and breach morphology are updated assuming vertical breach slopes and constant lateral erosion depth ($db$) concentrated at the breach bottom (Figure 3.15). Since the selected model for sand erosion only calculates the eroded breach cross-section area ($dA_b$), the ratio between lateral and vertical erosion ($c_{db/dz,b}$) have to be assumed and is given as input.
Based on laboratory tests (Hassan et al., 1999), possible values are suggested in the model:

\[
c_{db/dz,b} = \frac{db/dt}{dz/dt}; \quad c_{db/dz,b} \approx 1
\]  

\[3.44\]

**a) Rectangular section**

\[dA_b = dz(B_b + 2db) \quad \Rightarrow \quad 2c_{db/dz,b}(dz)^2 + B_b dz - dA_b = 0\]

\[dz = \frac{-B_b + \sqrt{B_b^2 + 8c_{db/dz,b}dA_b}}{4c_{db/dz,b}}\]

**b) Non-rectangular section**

\[dA_b = dz(B_b + 2db) + 2db \cdot dz_c \quad \Rightarrow \quad 2c_{db/dz,b}(dz)^2 + (B_b + 2c_{db/dz,b}dz_c)dz - dA_b = 0\]

\[dz = \frac{-b + \sqrt{b^2 + 8c_{db/dz,b}dA_b}}{8c_{db/dz,b}}; \quad b = B_b + 2c_{db/dz,b}dz_c\]

Figure 3.15: Sand core erosion depths and breach morphology

**Breach slopes instability** is treated in the model with a conventional *limit equilibrium approach* assuming a vertical failure surface characterised by scour erosion and shear failure, which is typical for non-cohesive materials (Figure 3.16). Suction pressure is neglected, the dike over the phreatic line is assumed dry and the wasted sediments are instantaneously removed from the breach. The *phreatic line* \((h_f)\) is calculated under the assumptions of a steady flow, homogeneous materials and an impermeable dike base (Casagrande, 1940; Darcy, 1856; Dupuit, 1863; Mishra & Singh, 2005).

The failing sand block slumps into the breach when the mobilising force \((F_{mo})\), i.e. shear stress, exceeds the resisting force \((F_{re})\) i.e. shear strength, giving a factor of safety (FOS) lower than 1:

\[\text{FOS} = \frac{F_{re}}{F_{mo}} < 1\]  

\[3.45\]

- Mobilising force \((F_{mo})\): \[F_{mo} = W - P_{at} + P_{dc} \tan \phi_s\]  

\[3.46\]

- Resisting force \((F_{re})\): \[F_{re} = R\]  

\[3.47\]

**Equilibrium final breach** is trapezoidal with slopes equal to the angle of repose of sand \((\phi_{0,s})\). It is set in the model at the end of phase 6 of Figure 3.10, when backwater level \((h_p)\) is equal to mean water level (MWL).
Weight of failing block \((W)\): \(W = W_1 + W_2 + W_3\)

**Phreatic line in the sand core (a)**
- Weight of dry clay: \(W_1 = \rho_{c,d}S_c c_{db/dz,b} \left( dz_c \right)\)
- Weight of dry sand: \(W_2 = \rho_{s,d} \left( S - S_c \right) c_{db/dz,b} \left( dz_c \right)\)
- Weight of saturated sand: \(W_3 = \rho_{s}g \left( H_b - S \right) c_{db/dz,b} \left( dz_c \right)\)

**Phreatic line in the clay cover (b)**
- Weight of dry clay: \(W_1 = \rho_{c,d}S_c c_{db/dz,b} \left( dz_c \right)\)
- Weight of saturated clay: \(W_2 = \rho_{c}g \left( S_c - S \right) c_{db/dz,b} \left( dz_c \right)\)
- Weight of saturated sand: \(W_3 = \rho_{s}g \left( H_b - S \right) c_{db/dz,b} \left( dz_c \right)\)

**Hydrostatic water pressures \((P)\)**
- Mobilising pressure: \(P_{de} = \rho_w g \left( H_b - S \right) c_{db/dz,b} \left( dz_c \right)\)
- Stabilising pressure: \(P_{st} = P_{st,1} + P_{st,2} = \rho_w g \left( H_b - S \right) c_{db/dz,b} \left( dz_c \right) + \rho_w g \left( h_f \right) c_{db/dz,b} \left( dz_c \right)\)

**Shear strength -Mohr-Coulomb criterion- \((R)\):**
\[ R = c_{eq} \left( H_b - \left( dz_c \right) \right) + N \tan \phi_s \]

**Weighed cohesion \((c_{eq})\):**
\[ c_{eq} = c_b \left( H_b - \left( dz_c \right) \right) \]

**Normal resisting force \((N)\):**
\[ N = P_{st,1} \tan \phi_s \]

Figure 3.16: Definition sketch of breach slopes instability

**Erosion of cover layer** at the dike crest (phase 4 of Figure 3.10) and the outer slope (phase 5 of Figure 3.10) is much slower than the erosion of the sand core and is therefore assumed negligible in the model (D’Eliso et al., 2006b). The cover layer at the dike crest is a vertical headcut with a base made of sand. In the model, the advance seaward of the cover layer \((dX_C)\) is proportional to the vertical erosion depth in the sand core at the cover layer base \((dz)\). The proportionality coefficient \(c_{dX_c/dz}\) can be tentatively assumed equal to 1+1.5:

\[ dX_C = c_{dX_c/dz} \cdot dz \quad \text{with} \quad c_{dX_c/dz} = 1+1.5 \quad \text{[3.48]} \]
At the outer slope, the dike cover fails being removed vertically \((dZ_c)\) together with vertical erosion of sand \((dz)\):

\[
dZ_c = dz
\]  
[3.49]

### 3.1.2.4 Uncertainties related to material properties

The morphodynamic module, although based on very simple models, requires several input parameters (grass, clay and sand parameters), which are generally associated with large uncertainties, due to the natural variability of material properties. Before stochastic morphodynamic models will be available, uncertainty analysis is the only way to understand how much they influence the model outcomes (Chapter 5).

Sediment density \((\rho)\) and size \((D_{50})\) are in principle the less uncertain parameters, because they should have known and controlled values of construction materials. However, they strongly affect the erosion process and have to be included in the uncertainty analyses. Other parameters, influenced by the weathering of grass and clay, like grass cover factor \((C_f)\), plasticity index \((I_p)\), liquid limit \((w_l)\) and unconfined compressive strength \((UCS)\) change over time and have a high natural variation both locally along the inner slope and more extensively along the dike line. Weathering of sand is relatively low due to the protective function of the cover layer. Input parameters also include the initial headcut height \((H_{H,0})\) and the initial breach channel width \((B_{B,0})\) that assume empirical and experience-based values.

**Model parameters** include:

- Total Manning’s roughness \((n_{tot})\): available formulae are generally not appropriate for small super-critical flow depths (Section 3.1.2.1) and are strongly dependent on the data set used for validation. In the preliminary model, it is however calculated and not given as input, because there are not precise indications on its possible values;
- Ratio between lateral and vertical erosion in the sand core \((c_{db/dz,b})\): suggested values are derived from results of laboratory tests on breaching. Nevertheless, due to the importance of this parameter and to the limited data set used as reference, the associated uncertainties have to be considered in the uncertainty analyses (Chapter 5).

### 3.1.3. Limitations of the mathematical formulation

The following limitations of the mathematical formulation are worth to be mentioned:

(i) Water infiltration in the dike, grass and clay cover instability are neglected;
(ii) Combined flow discharge \( (q_{\text{Comb}}) \) is calculated based on a very simple empirical model by Bleck et al. (2000) which is further reviewed and modified in the present model (Section 3.1.1.3);

(iii) Breach initiation is oversimplified;

(iv) Grass erosion is not progressive, but calculated as function of a threshold value of the cumulated effective bottom shear stress \( (\Sigma \tau_{0,e}) \) and a conventional erosion depth at failure \( (\Delta z_g) \) is defined;

(v) Models for the calculation of total Manning’s roughness \( (n_{\text{tot}}) \) are not derived for small depth super-critical flows;

(vi) Scour at the headcut base is not fully described, but only erosion at the headcut base is included;

(vii) Headcut advance is calculated with a continuous averaged model;

(viii) Sediment transport formulae used for sand erosion are not strictly valid during breaching;

(ix) Breach slopes instability is oversimplified and doesn’t include the suction pressure in the dike;

(x) Dike base is assumed to be erosion-resistant.

### 3.2. Model implementation

The preliminary model is based on the set of equations which have been selected and described in Section 3.1 (Table 3.7).

**Table 3.7: Model and equations adopted in the preliminary model**

<table>
<thead>
<tr>
<th>Phases</th>
<th>Models and equations</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading cases (Section 3.1.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wave overtopping</td>
<td>Schüttrumpf formulae</td>
<td>Figure 3.4</td>
</tr>
<tr>
<td>Combined flow</td>
<td>Steady non-uniform free surface flow equations</td>
<td>Figure 3.7</td>
</tr>
<tr>
<td>Overflow</td>
<td>Weir - Contracted flow formulae</td>
<td>Figure 3.8</td>
</tr>
<tr>
<td></td>
<td>eqs. [3.1]-[3.4]</td>
<td>eqs. [3.11]</td>
</tr>
<tr>
<td></td>
<td>eq. [3.11]</td>
<td></td>
</tr>
<tr>
<td>Loading cases (Section 3.1.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phase 1 Grass erosion</td>
<td>Effective bottom shear stress</td>
<td>eqs. [3.15]-[3.20]</td>
</tr>
<tr>
<td>Phase 2 Local clay erosion</td>
<td>Excess bottom shear stress</td>
<td>eqs. [3.21]-[3.24]</td>
</tr>
<tr>
<td>Phase 3a Headcut erosion</td>
<td>Excess bottom shear stress</td>
<td>eq. [3.25]</td>
</tr>
<tr>
<td>Phase 3b Sudden cover failure</td>
<td>Continuous advance model</td>
<td>eqs. [3.27]-[3.34]</td>
</tr>
<tr>
<td>Breaching phases (Section 3.1.2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phase 4 Sand erosion-crest</td>
<td>Sediment transport formulae:</td>
<td>eqs. [3.36]-[3.49]</td>
</tr>
<tr>
<td>Phase 5a and 5b Sand erosion-seaside</td>
<td>Bagnold-Visser or Yang</td>
<td>Table 3.5</td>
</tr>
<tr>
<td>Phase 6a and 6b Sand erosion Full breach</td>
<td>Breach morphology:</td>
<td>Table 3.6</td>
</tr>
<tr>
<td></td>
<td>Breach slopes instability:</td>
<td>Figure 3.15</td>
</tr>
<tr>
<td></td>
<td>parametric relations</td>
<td>Figure 3.16</td>
</tr>
<tr>
<td></td>
<td>limit equilibrium approach (shear)</td>
<td></td>
</tr>
</tbody>
</table>
A prototype sea dike (Figure 3.17) is used for model application (Section 3.2.2), for comparison with the detailed model (Chapter 4) and for the uncertainty analyses (Chapter 5). A complete list of input parameters is given in Annex B.

![Figure 3.17: Prototype dike for model application, comparative and uncertainty analyses](image)

### 3.2.1. Modules of the preliminary model

The preliminary model has a modular structure, including the following modules (Figure 3.18):

- **Input module** reads the input parameters, calculates the time series of waves and water levels, material properties and defines calculation dike sections;

![Figure 3.18: Simplified flow chart and modules of the preliminary model](image)
Chapter 3  Preliminary model: development, implementation and tentative validation

- **Hydrodynamic module** solves wave overtopping, overflow or combined flow equations (Section 3.1.1);
- **Morphodynamic module** solves erosion and instability models for grass, clay and sand, according to the six breaching phases described in Figure 3.10 (Section 3.1.2);
- **Output module** saves model outcomes at fixed time steps and at the end of each phase.

For each phase, the hydrodynamic and morphodynamic modules are iteratively applied until conditions for the successive phase are verified (Figure 3.18).

### 3.2.2. Computational aspects, results and discussion

#### 3.2.2.1 Computational aspects

The input module defines calculation sections at the outer slope, at the dike crest and at the inner slope (Figure 3.19). Four main sections are fixed defining three segments (seaside, dike crest and landside). At each segment, calculation sections are generated by the model according to the specified grid spacing ($\Delta x$).

The grid spacing ($\Delta x$) is given as input and there are no restrictions due to numerical instability problems during clay cover erosion (phases 1-3 of Figure 3.10). During sand core erosion (phases 4-6 of Figure 3.10), the Courant number ($C$) of the flow should be limited and possibly lower than 1 (Figure 3.19).

The time step ($\Delta t$) is calculated in the model depending on the cause of breach initiation:

- Wave overtopping or combined flow: it is dynamic and given by the ratio between each wave period ($T$) of the time series of waves, and the number of time steps within a wave cycle $N_{pw}$ (Section 3.1.1);
- Overflow: it is constant during the simulation and given as input.

![Figure 3.19: Definition of calculation sections, grid spacing ($\Delta x$) and time step ($\Delta t$)](image)

- Courant number: $C = \frac{c\Delta t}{\Delta x} \leq 1$
- Flow velocity: $c = \sqrt{gh}$
- Flow depth: $h = h_w = \frac{2}{3} MWL$

#### 3.2.2.2 Computational results: analysis

Application of the model to the prototype dike (Figure 3.17), for different loading cases (wave overtopping, combined flow and pure overflow) in case of
irregular waves provides typical model outcomes and show the influence of waves on the breaching process (Table 3.8).

Table 3.8 Application of the preliminary model to prototype dike: outcomes

<table>
<thead>
<tr>
<th>Outcome</th>
<th>Symbol</th>
<th>SI unit</th>
<th>WOIW</th>
<th>CFIW</th>
<th>OVIW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit discharge ($t \leq t_f$)</td>
<td>$q_{in} = V_{Ot}/(B_0 t_f)$</td>
<td>[m$^3$/sm]</td>
<td>0.033</td>
<td>0.094</td>
<td>0.019</td>
</tr>
<tr>
<td>Peak outflow discharge</td>
<td>$Q_{O,P}$</td>
<td>[m$^3$/s]</td>
<td>1192</td>
<td>1162</td>
<td>1291</td>
</tr>
<tr>
<td>Total released water volume</td>
<td>$V_{Ot_{ot}}$</td>
<td>[m$^3$]</td>
<td>650417</td>
<td>705387</td>
<td>705019</td>
</tr>
<tr>
<td>Time of grass failure</td>
<td>$t_{gf}$</td>
<td>[hr]</td>
<td>3.63</td>
<td>0.58</td>
<td>1.48</td>
</tr>
<tr>
<td>Time of headcut initiation</td>
<td>$t_{H}$</td>
<td>[hr]</td>
<td>4.39</td>
<td>1.21</td>
<td>3.15</td>
</tr>
<tr>
<td>Time of cover erosion</td>
<td>$t_{ce}$</td>
<td>[hr]</td>
<td>5.71</td>
<td>2.09</td>
<td>4.63</td>
</tr>
<tr>
<td>Time of cover failure</td>
<td>$t_{cf}$</td>
<td>[hr]</td>
<td>5.84</td>
<td>2.14</td>
<td>4.87</td>
</tr>
<tr>
<td>Time of dike failure</td>
<td>$t_{df}$</td>
<td>[hr]</td>
<td>6.03</td>
<td>2.37</td>
<td>5.07</td>
</tr>
<tr>
<td>Erosion rate (phase 2)</td>
<td>$dz/dt$</td>
<td>[mm/s]</td>
<td>0.07</td>
<td>0.16</td>
<td>0.04</td>
</tr>
<tr>
<td>Erosion rate (phase 3a)</td>
<td>$dz/dt$</td>
<td>[mm/s]</td>
<td>0.08</td>
<td>0.14</td>
<td>0.07</td>
</tr>
<tr>
<td>Headcut height growth rate</td>
<td>$dH/H/dt$</td>
<td>[mm/s]</td>
<td>0.13</td>
<td>0.24</td>
<td>0.15</td>
</tr>
<tr>
<td>Headcut advance rate</td>
<td>$dX/dt$</td>
<td>[mm/s]</td>
<td>0.34</td>
<td>0.66</td>
<td>0.41</td>
</tr>
<tr>
<td>Initial breach channel width</td>
<td>$B_{0,0} = B_{H}$</td>
<td>[m]</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Final breach width</td>
<td>$B_b$</td>
<td>[m]</td>
<td>58.69</td>
<td>50.13</td>
<td>56.38</td>
</tr>
</tbody>
</table>

WOIW: Wave overtopping ($H_s = 3.00$ m, $T_P = 10.0$ s, SWL = 6.50 m)  
CFIW: Combined flow ($H_s = 3.00$ m, $T_P = 10.0$ s, SWL = 7.05 m)  
OVIW: Overflow ($H_s = 0.00$ m, $T = 0.0$ s, SWL = 7.05 m)  

The analysis of the outcomes shows encouraging results with respect to the breach outflow hydrograph, the time associated to breaching, the grass and clay erosion, the headcut advance and the breach widening (see Figure 3.10). Model outcomes in case of wave overtopping are shown in Figure 3.20-Figure 3.23:

(i) **Breach outflow hydrograph** (phases 1-6 of Figure 3.10)  
Breach outflow hydrograph ($Q_b(t)$) has three different regions (Section 2.2.1 and Figure 3.20a):

- **Region 1 - Wave overtopping** (Figure 3.20b): the discharge due to waves only ($q$) is negligible;
- **Region 2 - Combined flow** (Figure 3.20c): the backwater level effect is negligible ($S_Q \approx 1$), with increasing breach outflow discharge ($Q_b$) and volume of water released in the polder ($V_O$);
- **Region 3 - Combined flow** (Figure 3.20c): the backwater level effect is not negligible ($S_Q < 1$), with decreasing breach outflow discharge ($Q_b$) and volume of water released in the polder ($V_O$).

(ii) **Time associated to breaching** (phases 1-6 of Figure 3.10)  
The failure of the sand core (phases 4-6) develops much faster ($t_{b-cf} \approx 5\%t_{cf}$) than the failure of the clay cover (phases 1-3). This difference in time scales increases
if wave overtopping discharge (q) decreases and the inner slope is not damaged at the beginning of sea storm (good grass conditions: \( C_f \approx 1 \)).

Looking at the breach outflow hydrograph \( Q_b(t) \), dike breaching is strictly not reversible after the threshold time \( t_t \) (Figure 3.20a). In practice, the dike is seriously vulnerable after the time of cover failure \( t_{cf} \), when the polder area is about to be flooded. Therefore, the warning time \( t_w \) may be defined as the time of cover failure \( t_{cf} \).

(a) Breach outflow hydrograph \( Q_b(t) \)

(b) Wave overtopping

(c) Combined flow

Figure 3.20: Breach outflow hydrograph \( Q_b(t) \) for wave overtopping (see Table 3.8)

(iii) Grass and clay erosion (phases 1-3a of Figure 3.10)
Grass erosion depth (\( \Delta z_g \)) focuses on the grass failure itself, without simulating the progressive failure over time (Section 3.1.2.1 and Figure 3.21).

During clay erosion, two regions with different erosion rates \( (dz/dt) \) are well defined (Figure 3.21). In both regions, cumulated erosion depth \( (\Sigma dz) \) is almost linearly growing, and the erosion rate \( (dz/dt) \) is lower during local clay erosion than during headcut erosion. In fact, it is physically reasonable that scour erosion induced by an impinging jet may be faster than local erosion due to canalized flow.

(iv) Headcut advance (phase 3a of Figure 3.10)
The growth of both headcut height \( (H_i) \) and cumulated headcut advance \( (\Sigma dX) \) is almost linear (Figure 3.22). This is in agreement with the evidence that the higher the headcut is, the more effective is the vertical erosion at the headcut base \( (dz) \) and the higher the headcut advance \( (dX) \).
Breaching of sea dikes initiated by wave overtopping  
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4. Breaching of sea dikes initiated by wave overtopping

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Figure 3.21: Cumulated erosion depth ($\Sigma dz$) in the clay cover for wave overtopping (see Table 3.8)

Figure 3.22: Headcut height ($H_H$) and cumulated headcut advance ($\Sigma dX$) for wave overtopping (see Table 3.8)

(v) Breach widening (phases 4-6 of Figure 3.10)
Breach channel width ($B_b$) starts growing after crest failure (Figure 3.23). As expected, most of the breach widening occurs after dike failure when the flow in the breach is transitional (phase 6$a$). When the flow is sub-critical everywhere in the breach channel (phase 6$b$), lateral erosion ($db$) and breach widening are negligible.
3.2.2.3 Computational results: effect of loading case

Comparison between simulated dike breaching initiated by wave overtopping, combined flow and overflow indicates (Table 3.8):

- Water discharge released over the dike before the threshold time ($t_t$) is very sensitive to the loading case: the averaged combined flow discharge is three times higher than the averaged wave overtopping discharge ($q_m$);
- Time associated to sand erosion and development of the final breach (difference between time of breaching $t_b$ and time of cover failure $t_{cf}$), peak outflow discharge ($Q_{b,p}$) and final breach width ($B_{b}$) are much less sensitive to the loading case because they depend more on the total volume of water realised in the polder ($V_{O,tot}$) and the mean water level (MWL) rather than on the additional action of waves;
- Although the averaged overtopping discharge ($q_m$) is higher than the overflow discharge ($q_{in}$), the time of cover failure ($t_{cf}$) and the time of breaching ($t_b$) are higher for wave overtopping than for overflow, due to the different nature of the flow over the dike (triangular over time for wave overtopping and constant for overflow), see Section 3.1.1;
- The peak outflow discharge ($Q_{b,p}$) is (i) higher for overflow than for wave overtopping, due to higher mean water level (MWL) and (ii) lower for combined flow than for overflow because waves may increase or decrease pure overflow discharge depending on the mutual influence of waves and water level (Section 3.1.1.3).
3.2.3. Tentative model validation

Appropriate laboratory and field tests for model validation are still not available, because previous investigations reproduce (i) dams that have different layouts and geometrical scale than sea dikes or sand dikes without clay cover, and (ii) breaching initiated by overflow. Only a set of tests on breaching of sand dikes with and without a clay cover initiated by wave overtopping is available (Geisenheiner & Kortenhaus, 2006).

Experienced dike failures have been generally qualitatively documented, but collection of data from floods of 1953 in The Netherlands (Rijkswaterstaat, 1961) and 1962 in Germany (Kolb, 1962) provide information at least for a tentative validation.

Appropriate near full scale laboratory tests will be performed at the end of this year by LWI in the Large Wave Flume of Hannover (Germany) and will be available in 2008 for final validation. At this stage, only tentative validation is therefore possible.

3.2.3.1 Selected data sets and validation procedure

Tentative validation is performed by using two data sources, for a total of five cases of sea dikes where breaching was initiated by wave overtopping:

(i) Laboratory tests at LWI, Germany (Geisenheiner & Kortenhaus, 2006)
A set of 11 small-scale tests on breaching of non-cohesive sea dikes initiated by wave overtopping has been performed in the wave flume of Leichtweiß-Institute for Hydraulic Engineering and Water Resources, TU Braunschweig (Germany). The last test was conducted introducing a clay cover on dike top. The wave flume is 90.00 m long, 2.00 m wide and 1.25 m high. A scaling factor of 1:10 was used in the tests. All tests were recorded with a video camera at the inner slope to better understand the physical processes involved and to estimate the breach width ($B_b$). Tests Nr. 10 and Nr. 11 are selected for tentative validation of the model, because dike breaching develops exactly in the centre of the flume and seepage flow effects on breaching are less important than in the other tests (Table 3.9).

(ii) Experienced dike failures during 1953’s flood, The Netherlands (Rijkswaterstaat, 1961)
Description of breaches opened during 1953’s flood in the Netherlands has been collected by Rijkswaterstaat, Ministry of Transport, Public Works and Water Management (The Netherlands). Dike height ($H_d$), mean water level over time ($MWL(t)$), time of breaching ($t_b$) and width of the final breach ($B_b$) are normally reported. Detailed information on wave climate, initial grass condition and material properties are however not available. Therefore, sea storm of constant significant wave height ($H_S$) and grass/clay of medium erosion resistance are assumed for simulation in the model. Three cases of dike failure are used for tentative validation (Table 3.10).
Time of cover failure ($t_{cf}$), calculated from indications of Rijkswaterstaat (1961) is the time interval between the initiation of the sea storm at the closest station where water levels were recorded ($t = t_0 = 0$) and time of the initial breach in the sand core of the dike ($t = t_{cf}$).

Table 3.9: Tested sea dikes used at LWI (2006)

<table>
<thead>
<tr>
<th>Test</th>
<th>Nr. 10: sand dike without clay cover</th>
<th>Nr. 11: sand dike with clay cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dike cross-sections</td>
<td><img src="image" alt="Dike Diagram" /></td>
<td><img src="image" alt="Dike Diagram" /></td>
</tr>
<tr>
<td>Test end</td>
<td>at flume sides</td>
<td>before materials mixing</td>
</tr>
<tr>
<td>Significant wave height $H_S$ [m]</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>Mean period $T_m$ [s]</td>
<td>2.25</td>
<td>2.25</td>
</tr>
<tr>
<td>Still water level $SWL$ [m]</td>
<td>0.47</td>
<td>0.50</td>
</tr>
<tr>
<td>Time of breaching $t_b$ [hr]</td>
<td>0.31</td>
<td>0.55</td>
</tr>
<tr>
<td>Final breach width $B_b$ [m]</td>
<td>1.60</td>
<td>0.92</td>
</tr>
</tbody>
</table>

Table 3.10: Selected sea dikes failed in 1953’s flood in The Netherlands

<table>
<thead>
<tr>
<th>Description</th>
<th>Polder Papendrecht</th>
<th>Herenpolder</th>
<th>Polder Zuidland</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean water level $MWL_{max}$ [m]</td>
<td>5.20</td>
<td>4.50</td>
<td>4.20* (3.75-4.20)</td>
</tr>
<tr>
<td>Dike height $H_d$ [m]</td>
<td>5.30</td>
<td>4.50* (4.30-4.70)</td>
<td>4.25* (4.10-4.45)</td>
</tr>
<tr>
<td>Crest width $B_d$ [m]</td>
<td>7.00</td>
<td>7.00</td>
<td>7.00</td>
</tr>
<tr>
<td>Outer slope $m$ [1]</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Inner slope $n$ [1]</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Polder area $A_p$ [m²]</td>
<td>94.4·10⁶</td>
<td>2.2·10⁶</td>
<td>0.9·10⁶</td>
</tr>
<tr>
<td>Sand sediment size $D_{50,s}$ [mm]</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Friction angle of sand $\phi_s$ [°]</td>
<td>32</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Type of widening</td>
<td>Limited</td>
<td>Non-limited</td>
<td>Non-limited</td>
</tr>
<tr>
<td>Type of base</td>
<td>Non-erodible</td>
<td>Non-erodible**</td>
<td>Non-erodible**</td>
</tr>
<tr>
<td>Time of cover failure $t_{cf}$ [hr]</td>
<td>19.50</td>
<td>19.75</td>
<td>20.50</td>
</tr>
<tr>
<td>Time of breaching $t_b$ [hr]</td>
<td>$t_{ce}+2.50$</td>
<td>$t_{ce}+0.75-1.75$</td>
<td>$t_{ce}+1.50$</td>
</tr>
<tr>
<td>Final breach width $B_b$ [m]</td>
<td>85 (b)-110 (t)</td>
<td>120</td>
<td>84</td>
</tr>
<tr>
<td>Final backwater level $h_p$ [m]</td>
<td>1.10</td>
<td>2.50</td>
<td>-</td>
</tr>
</tbody>
</table>

* Selected, ** Assumed, b Bottom, t Top

**Tentative validation** of the model includes comparison between simulated and observed (i) time of breaching ($t_b$), (ii) time of cover failure ($t_{cf}$) in case of sand dikes with cover layer (Figure 3.24a) and (iii) final breach width ($B_b$) or
width of the initial breach channel \( (B_{b,0}) \) for laboratory test Nr. 11 (Figure 3.24b). Comparison of the peak outflow discharge \( (Q_{b,p}) \) is not possible, because it was not recorded.

### 3.2.3.2 Validation results and discussion

Simulated versus measured parameters for the five selected cases indicate different levels of accuracy of the model. The preliminary model generally predicts faster time associated to breaching (Figure 3.24a) and a larger breach width (Figure 3.24b) than the available data:

1. **Laboratory test Nr. 10 (sand dike without clay cover):** time of breaching \( (t_b) \) is underestimated (19%) and final breach width \( (B_b) \) is overestimated (7%). Breach parameters are not compared at the end of the process, but when sides of the flume start influencing erosion, which is during phase 6 of the model (Figure 3.10).

2. **Laboratory test Nr.11 (sand dike with clay cover):** the test (i) doesn’t include clay and headcut erosion (phases 2 and 3a of Figure 3.10), because when overtopping starts, clay cover has already failed at the inner toe due to seepage, (ii) shows headcut erosion of sand-clay scour up to the formation of the initial breach channel in the sand core (phases 3b, c of Figure 3.10) and (iii) stops before sand core erosion (phases 4-6 of Figure 3.10) in order to avoid mixing of sand and clay. The preliminary model doesn’t simulate the headcut erosion in sand-clay scour, but (i) it assumes an instantaneous failure of cover layer as soon as sand is exposed to the flow action and (ii) it empirically calculates the width of the initial breach channel \( (B_{b,0}) \), which is highly underestimated, if compared with the measured datum (80%). The scatter of the results is sensibly reduced in the detailed model (Figure 4.19);  

3. **Polder Papendrecht (sea dike):** time of cover failure \( (t_{cf}) \) and time of breaching \( (t_b) \) are underestimated (respectively 37% and 42%). The simulation is stopped when the breach width \( (B_b) \) is equal to its maximum admissible value, determined by the lateral heads of the dike that impose a limitation on further widening. Final breach width \( (B_b) \) is therefore not compared, but imposed.

4. **Herenpolder (sea dike):** time of cover failure \( (t_{cf}) \) and time of breaching \( (t_b) \) are underestimated (respectively 29% and 27%). Final breach width \( (B_b) \) is overestimated (22%);  

5. **Zuidland (sea dike):** time of cover failure \( (t_{cf}) \) and time of breaching \( (t_b) \) are underestimated (respectively 24% and 26%). Final breach width \( (B_b) \) is overestimated (43%).

A summary of results of the tentative validation of the preliminary model and a comparison with the tentative validation of the detailed model are provided in Figure 4.20 and Table 4.11.
Chapter 3 Preliminary model: development, implementation and tentative validation

3.3. Capabilities and limitations of the preliminary model

The preliminary model, as a first step of a model system for the breaching of sea dikes initiated by wave overtopping, overflow or a combination of both, provides an overview of the processes involved and indicates priority issues to be pursued in the detailed model in order to achieve a more process-oriented description of the breach development.

The model, based on simple formulae and equations, calculates breach initiation and growth from the initial to the final breach and is the first model which applies to sand-clay dikes with grass and which includes wave action as a primary load (Section 3.1). Results from the model are qualitatively reliable. Tentative validation against laboratory tests and experienced dike failures shows qualitatively reasonable agreement between measured and simulated breach parameters. The scatter is generally lower than 30% for both time associated to

![Figure 3.24: Preliminary model validation: simulated versus observed breach parameters](image)

The scatter between measured and simulated values is relatively limited (Figure 3.24), if compared with the uncertainties of:

(i) **Laboratory tests - model inputs**: the influence of seepage and sides of the flume are not exactly quantified in the tests and neglected in the model, effective size of polder area ($A_p$) have not been measured in the tests and therefore roughly assumed equal to the portion of the flume behind the dike in the model;

(ii) **Experienced dike failures - model inputs**: time of cover failure ($t_{cf}$) and final breach width ($B_b$) are measured by eye-witnesses, wave climate, grass and clay properties have typical and not site specific values in the model.
breaching and breach width. Appropriate data for final validation are still not available (Section 3.2.3).

Model limitations are a direct consequence of the simplified equations and assumptions used to describe the processes involved:

1. Breach initiation is simply randomly located at the inner slope;
2. Water infiltration, grass and clay instability mechanisms are completely neglected;
3. The headcut advance model is continuous and not discrete;
4. Sediment transport models are strictly not completely valid during dike breaching.

In the determinant phases of the breaching process, namely grass and clay erosion and failure (phase 1-3 of Figure 3.10), the following aspects are particularly important:

- The effect of wave action on the breaching process concentrates during the erosion of clay cover with grass, while it significantly decreases and becomes negligible during sand erosion, where combined flow and pure overflow dominate;
- An indicator of the dike resistance against sea storms, useful for flood risk management, is the warning time ($t_w$), which may be defined as the time of cover failure ($t_{cf}$).

Therefore, the detailed model should particularly concentrates on the simulation of grass and clay erosion, including new processes and improving single aspects already considered in the preliminary model (Chapter 4).
4. Detailed model: development, implementation and tentative validation

The detailed model is the second part of a model system for dike breach modelling (Section 2.5), based on the results of the preliminary model (Chapter 3). The main objective is to improve the prediction capability of dike breaching looking to a more process-oriented description of the breach growth. The model includes all relevant processes and failure mechanisms involved during dike breaching initiated by wave overtopping, is based on simple equations, but some assumptions imposed in the preliminary model are removed. Like preliminary model, the detailed model has a modular structure (Figure 4.1).

The hydrodynamic module (Figure 4.1a and Section 4.1.1) includes free surface flow (wave overtopping, combined flow and overflow at the dike and through the breach) and water infiltration in the dike induced by the free surface elevation and flow.

The morphodynamic module (Figure 4.1b and Section 4.1.2) includes grass, clay and sand erosion and mass instability, according to a sequence of six breaching phases (Section 4.1.2), based on the preliminary model results (Section 3.1.2). Compared to the preliminary model, the detailed model:

1. Introduces the following new processes: (i) water infiltration in the dike, (ii) turf set-off, (iii) clay cover sliding and up-lift and (iv) headcut in sand-clay scour;

2. Improves the description of the following processes: (i) wave overtopping, (ii) breach initiation, (iii) headcut erosion and advance.

Model uncertainties related to input parameters (Sections 4.1.1.3 and 4.1.2.5) are still important as in the preliminary model (Sections 3.1.1.4 and 3.1.2.4).

In Figure 4.1, the shape of the boxes indicates different types of information: (i) slightly rounded: simulated process and approach, (ii) rectangular: selected equations with references and main hypotheses (bold font), and referring models, (iii) rectangular without a corner: hypotheses of present model, (iv) rounded shape: new aspects.
Breaching of sea dikes initiated by wave overtopping

C. D’Eliso

Figure 4.1: Structure of the detailed model: hydrodynamic and morphodynamic modules
In this Chapter, the development, implementation and tentative validation of the detailed model are addressed, but more details are provided in D’Eliso et al. (2006a):

- Mathematical formulation: description of improvements of simulated processes and uncertainties (Section 4.1);
- Model implementation: description of modules, discussion of results and tentative validation (Section 4.2);
- Capabilities and limitations of the model (Section 4.3).

### 4.1. Mathematical formulation

The mathematical formulation of the dike breaching implemented in the preliminary model (Chapter 3) is completed and improved in the detailed model, trying to better simulate the complex 3D physical process of breaching (Chapter 2). The detailed model still simulates a simplified breaching process based on the following assumptions:

- **2D + 2D process**: both cover and core erosion and failure are calculated with 2D models and parametric equations, based on simple results of existing models, to also include width of the headcut scour in the clay cover and of the breach channel;
- Water infiltration in the dike due to high mean water level in the sea is assumed to have a secondary effect on breaching initiated by wave overtopping;
- Hydrodynamic and morphodynamic modules are not coupled.

Only improvements of the hydrodynamic and morphodynamic description are presented in this section, while the full description is provided in Section 3.1.

#### 4.1.1. Improvements of the hydrodynamic module

The hydrodynamic module includes:

(i) Free surface flow along the dike profile and through the breach, mostly responsible for dike erosion;

(ii) Water infiltration in the dike induced by wave overtopping, combined flow or overflow at the inner slope, that leads to grass set-off and clay cover sliding and up-lift.

The hydrodynamic module uses the following inputs: wave climate at the dike toe (Section 3.1.1), initial volumetric water content in the dike ($\theta_i$) and other material properties related to water infiltration, such as saturated hydraulic conductivity ($k_s$), saturated ($\theta_s$) and residual ($\theta_r$) volumetric water content.

The selected models of the hydrodynamic module calculate:

- **Free surface flow**: flow velocity ($v$), flow depth ($h$), discharge over the dike ($q$) or through the breach ($Q_b$) and backwater level ($h_p$);
- **Water infiltration in the dike**: saturated water front \((z_s)\), infiltration water front \((z_w)\), volumetric water content in the dike \((\theta)\) and suction pressure \((u)\).

### 4.1.1.1 Free surface flow

Flow velocity \((v)\) and depth \((h)\) along the dike profile are optionally calculated by Schütrumpf’s formulae (Section 3.1.1.1) or by applying the Volume of Fluid (VOF) Model developed at Cornell University (Liu & Lin, 1997), i.e. COBRAS Model. This model is a 2DV solver of Reynolds-Averaged Navier-Stokes (RANS) equations, with a nonlinear turbulence model (Table 4.1). It is able to simulate propagation and breaking of regular waves also in presence of obstacles, i.e. structures (Liu & Lin, 2003). All details about the model are given in Liu & Lin (1997).

#### Table 4.1: Basic equations of RANS-VOF Model COBRAS (Liu & Lin, 1997)

**Reynolds-Averaged Navier-Stokes (RANS) equations:**

- Continuity equation: 
  \[
  \frac{\partial (u_j)}{\partial x_j} = 0 \quad j=1,2,3
  \]

- Momentum equation: 
  \[
  \frac{\partial (u_j)}{\partial t} + (u_i u_j) \frac{\partial (u_j)}{\partial x_i} = -\frac{1}{\rho} \frac{\partial (p)}{\partial x_j} + g_j + \frac{1}{\rho} \frac{\partial (\tau_{ij})}{\partial x_j} \quad j=1,2,3
  \]

  \(\tau_{ij}\) = viscous stresses, \(p\) = flow pressure

**VOF function \((F)\) for the free surface:**

\[
\frac{\partial F}{\partial t} + \frac{\partial}{\partial x_i}(u_i F) + \frac{\partial}{\partial x_j}(u_j F) = 0 \quad \text{with cell density} \quad \rho(x,z,t) = F(x,z,t)\rho_w
\]

Model outcomes include horizontal \((v)\) and vertical \((w)\) flow velocity, flow depth \((h)\), flow pressure \((p)\) and turbulence \((k)\). An example of velocity field \((v)\) and water surface \((\eta)\) calculated by the model is in Figure 4.2.

The available version of the model assumes a constant still water level (SWL) throughout the whole computational domain. Therefore, it is not possible to properly simulate the flow at the inner slope, but only up to the dike crest (section A in Figure 4.2). As in the preliminary model, flow velocity \((v)\) and depth \((h)\) at the inner slope are calculated with the Schütrumpf’s formulae using the following inputs: vertically-averaged horizontal flow velocity \((v_c)\) and depth \((h_c)\) from COBRAS Model.

Results of the RANS-VOF Model at the dike crest (Figure 4.3) obtained with the input parameters of Figure 4.2, indicate that both flow velocity \(v\) (Figure 4.3a), and discharge \(q\) (Figure 4.3b), approximately have a triangular shape, slightly asymmetrical, with (i) an almost vertical increasing segment up to the peak and (ii) a rapidly sloped decreasing segment. Therefore, the assumption of the preliminary model of a triangular shape of the discharge \((q)\) within a wave cycle.
seems to be reasonable, although the overtopping flow at the dike crest lasts a shorter time than the associated wave period $T$ (Section 3.1.1). Due to the interaction between waves, the overtopping flow of a regular wave is not constant, but changes from one wave to the next (Figure 4.3).

<table>
<thead>
<tr>
<th>Dike geometry</th>
<th>Dike height ($H_d$)</th>
<th>Crest width ($B_d$)</th>
<th>Outer slope (m)</th>
<th>Inner slope (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.00 m</td>
<td>3.00 m</td>
<td>6</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sea parameters</th>
<th>Significant wave height ($H_s$)</th>
<th>Peak period ($T_p$)</th>
<th>Mean water level (MWL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00 m</td>
<td>8.00 s</td>
<td>5.50 m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Numerical parameters</th>
<th>Grid spacing ($\Delta x$)</th>
<th>Grid spacing ($\Delta z$)</th>
<th>Time step ($\Delta t$)</th>
<th>Simulation time ($t_s$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15 m</td>
<td>0.075 m</td>
<td>0.1 s</td>
<td>100 s</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.2: Application of RANS-VOF model COBRAS

Significant improvements of wave overtopping simulation are achieved with the RANS-VOF model:

- The model is process-oriented solving governing equations of flow dynamics instead of empirical equations;
- The interaction between waves is included;
- The shape of the overtopping discharge ($q$) and flow ($v, h$) within a wave cycle is not assumed, but calculated.

Further significant improvement should be directed toward the extension of the RANS-VOF Model to 3D.

### 4.1.1.2 Water infiltration in the dike

Water infiltration in the dike is a time-space dependent 3D process and involves non-homogeneous materials. It has four sources (Section 2.1.3) and represents one of the main causes of breach initiation because it reduces the strength of the dike against (i) turf set-off, clay cover sliding and up-lift and (ii) inner slope instability (Section 2.2.5 and Figure 4.4). Moreover, the infiltration influences headcut advance and breach slopes instability because it modifies the volumetric water
content ($\theta$) and the suction pressure ($u$) in the dike. The presence of entrapped air regions below the dike crest may also be responsible for the formation of cracks that may lead to breaching (Elela, 1996; Zaradny, 1994).

Water infiltration in the dike is calculated in the model as a 2D process by assuming that:
- The calculation of the free surface flow and infiltration is not coupled;
- Wave run-up and run-down do not influence dike breaching initiated at the inner slope and are therefore neglected;
- The influence of rain may be included in the model through an ad-hoc initial volumetric water content ($\theta_i$), but it is not really simulated;
- The outer slope of the dike is assumed intact, i.e. without cracks. Therefore, infiltration due to high mean water level is very slow and is neglected in the model. A simple technique to include it, has however been presented in D’Eliso et al. (2006a). This technique is based on Darcy’s law.
for saturated soils and Wang Q.’s model (Wang et al., 2002) for unsaturated soils;

- The presence of entrapped air regions is not included;
- Infiltration due to wave overtopping and combined flow is assumed vertical.

All selected infiltration models are simplified solutions of Richard’s equation for unsaturated soils and depend on several soil parameters.

**a) Richard’s equation**

Richard’s equation (Richard, 1931), based on Darcy’s law, is the continuity equation of the flow in the soil (Table 4.2): flow head ($\Psi$) and boundary flux ($Q$) that are entering and leaving an element soil volume at a given time, equals the variation over time of the volumetric water content ($\theta$).

\[
\frac{\partial}{\partial t} \left( k \frac{\partial \Psi}{\partial x} \right) + \frac{\partial}{\partial y} \left( k \frac{\partial \Psi}{\partial y} \right) + Q = \frac{\partial \theta}{\partial t}; \quad k = (k_s, k_r); \quad \Psi = z + \frac{u}{\rho_s g}
\]

**Table 4.2: Richard’s equation (Richard, 1931)**

<table>
<thead>
<tr>
<th>Richard’s equation: [ \frac{\partial}{\partial t} \left( k \frac{\partial \Psi}{\partial x} \right) + \frac{\partial}{\partial y} \left( k \frac{\partial \Psi}{\partial y} \right) + Q = \frac{\partial \theta}{\partial t}; \quad k = (k_s, k_r); \quad \Psi = z + \frac{u}{\rho_s g} ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic conductivity ($k$): [ k(\Psi) = (k_s(\Psi), k_r(\Psi)) ]</td>
</tr>
<tr>
<td>Volumetric water content ($\theta$): [ \theta = \frac{V_w}{V_s} = \frac{\rho_w \theta}{\rho_s} ] Volume of water, $V_s$: Volume of sediments</td>
</tr>
</tbody>
</table>

Volumetric water content ($\theta$) is most used in soil science, while gravimetric water content ($w$) is more commonly used in geotechnical engineering practice.

**b) Soil parameters**

Both full and simplified solutions of Richard’s equation depend on two soil functions:

- Hydraulic conductivity function ($k$-u curve);
- Soil-water characteristic curve or soil-water retention curve ($\theta$-u curve), see Table 4.4: relationship between the volumetric water content ($\theta$) and the suction pressure ($u$), when the volumetric water content ($\theta$) is decreasing from the saturated value ($\theta_s$) to the residual value ($\theta_r$), i.e. desorption curve. A detailed review of empirical relations is given in Fredlung & Xing (1994).

Both functions include soil parameters, all empirically determined according to procedures described in the literature (Tzimopoulos et al., 2005; Wang et al., 2003; Weißmann, 2003). Typical values are given in Weißmann (2003) and EPA (1998), without any further explanation on their selection and use, but more detailed information can be found in the original references. If specific in-situ measurements are not available, typical values are used in the model:

**(i) Saturated hydraulic conductivity, i.e. soil permeability ($k_s$):** it increases from clay to sand soils (Rawls et al., 1992; Weißmann, 2003). The effect of grass on saturated hydraulic conductivity ($k_s$) is due to:
• Variation in soil pore distribution
• Development of preferential infiltration pathways due to grass roots.

The following empirical formulae may be used for the saturated hydraulic conductivity of grass ($k_{s,g}$):

- Holtan (1961):
  \[ k_{s,g} = \left( k_{s,c} + 2.78 \cdot 10^{-7} \cdot a^{1.4} \right) \text{m/s; } [a] = [%] \]  \[ [4.1] \]
  The percentage basal area of vegetation ($a$) depends on the type of grass. A value of $a = 40\div50 \%$ is suggested for grass used as dike protection (Holtan, 1961; Styczen & Morgan, 1995), which provides a saturated hydraulic conductivity of grass ($k_{s,g}$) in the range $\left(4.86\div6.66\right) \cdot 10^{-5}$.

- Pilarczyk (2003):
  \[ k_{s,g} = 10^{-5} \text{ m/s} \]  \[ [4.2] \]

Assuming a saturated hydraulic conductivity of clay ($k_{s,c}$) around $10^{-7}\div10^{-8}$ m/s, the saturated hydraulic conductivity of grass ($k_{s,g}$) is 2÷3 orders of magnitude higher (around $10^{-5}$ m/s), according to both eqs. [4.1] and [4.2].

Saturated hydraulic conductivity ($k_s$) in the dike is set according to the behaviour of a series system:

- Grass cover: $k_s = k_{s,c}$
- Clay cover and sand core: $k_s = \min\left(k_{s,c}, k_{s,g}\right) = k_{s,c}$  \[ [4.3] \]

(ii) Saturated volumetric water content ($\theta_s$): water content at saturation. It normally decreases from clay to sand soils (Brakensiek et al., 1981; Carsel & Parrish., 1988; Pajian, 1987; Weißmann, 2003).

(iii) Residual volumetric water content ($\theta_r$): water content for which a large suction range is required to remove additional water from the soil. It decreases from clay to sand soils (Brakensiek et al., 1981; Carsel & Parrish., 1988; Pajian, 1987; Weißmann, 2003).

(iv) Initial volumetric water content ($\theta_i$): water content at the beginning of the simulation ($\theta_i \in [\theta_r, \theta_s]$). It depends on initial dike conditions (site-specific value). Factors influencing initial dike conditions are rain, temperature and wave climate. If specific field measurements are not available, three classes of initial volumetric water content ($\theta_i$) are suggested (Table 4.3).

<table>
<thead>
<tr>
<th>Soil conditions</th>
<th>Initial volumetric water content ($\theta_i$) [m$^3$/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry (high temperature, low rain rate)</td>
<td>$\theta_i = \theta_r + 0.3(\theta_s - \theta_r)$</td>
</tr>
<tr>
<td>Medium wet</td>
<td>$\theta_i = \theta_r + 0.3(\theta_s - \theta_r) + 0.6(\theta_s - \theta_r)$</td>
</tr>
<tr>
<td>Wet (low temperature, high rain rate)</td>
<td>$\theta_i = \theta_r + 0.6(\theta_s - \theta_r)$</td>
</tr>
</tbody>
</table>

Table 4.3: Initial volumetric water content ($\theta_i$)

(v) Empirical parameters of Brooks and Corey’s curve (Table 4.4a):

- **Pore-size distribution index ($N_{BC}$)**: it summarises information about the pore size distribution in the soil. It increases from clay to sand soils (Brakensiek et al., 1981; Carsel & Parrish., 1988; Pajian, 1987);
• **Air entry value or bubbling suction head** ($\Psi_{B\text{BC}}$): matrix suction head where air starts entering the largest pores in the soil. It decreases from clay to sand soils.

(vi) **Empirical parameters of Van Genuchten’s curve -no physical meaning-** (Table 4.4b):

- **Inverse of air entry value or bubbling suction head** ($\alpha_{v\text{G}}$): decreases from clay to sand soils (Simůnek et al., 1986);
- **Pore-size distribution index parameter** ($n_{v\text{G}}$): it increases from clay to sand soils (Simůnek et al., 1986).

### Table 4.4: Soil-water characteristic curves

<table>
<thead>
<tr>
<th>Soil-water characteristic curves: $u = ρ_{\text{w}}g\Psi(\theta)$, $\theta \in [\theta_s, \theta_i]$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a) Brooks and Corey (1964):</strong> $\left(\frac{\theta - \theta_i}{\theta_s - \theta_i}\right) = \left(\frac{0.01}{\Psi_{B\text{BC}}}\right)^{-N_{B\text{BC}}} \Rightarrow \Psi(\theta) = 100 \cdot \Psi_{B\text{BC}} \left(\frac{\theta - \theta_i}{\theta_s - \theta_i}\right)^{-1/N_{B\text{BC}}}$</td>
</tr>
<tr>
<td><strong>b) van Genuchten (1980):</strong> $\left(\frac{\theta - \theta_i}{\theta_s - \theta_i}\right) = \left[1 + (0.01\alpha_{v\text{G}}\Psi)\right]^{-m} \Rightarrow \Psi(\theta) = 100 \cdot \frac{1}{\alpha_{v\text{G}}} \left[\left(\frac{\theta - \theta_i}{\theta_s - \theta_i}\right)^{-1/m} - 1\right]^{1/n_{v\text{G}}}$</td>
</tr>
</tbody>
</table>

m = \frac{1}{1 - 1/n_{v\text{G}}}  
From soil-water retention data  
Mualem’s model

When volumetric water content ($\theta$) is close to its saturated value ($\theta_s$), large deviations between the two curves are noticed (van Genuchten, 1980).

### c) Simplified solutions of Richard’s equation

Available **simplified solutions** of Richard’s equation are usually derived for rain, but two models are available for wave overtopping or combined flow (Wang, 2000a; Weißmann, 2003). Both of them only include the calculation of the infiltration in saturated soils, but in the detailed model they are extended to also calculate the infiltration in unsaturated soils (Wang et al., 2003; Wang, 2000a), the water content in the dike (Wang et al., 2003) and the suction pressure (Brooks & Corey, 1964; van Genuchten, 1980):

(i) **Saturated and infiltration water fronts**

Totally, three models have been selected for the use in the detailed model:

- **Weißmann model** (Weißmann, 2003)

It is derived from laboratory tests with wave overtopping and overflow at sea dikes with a clay cover without grass (Figure 4.4a). The model enables to calculate the saturated water front ($z_s$) solving Darcy’s law and assuming: (i) homogeneous and isotropic material with constant saturated hydraulic conductivity ($k_s$), (ii) hydraulic conductivity ($k(\Psi)$) is constant in the clay cover and equal to its saturated value ($k_s$), (iii) saturated soil, (iv) effects of wave overtopping and combined flow are included in a constant mean overflow depth ($\bar{h}$) over the simulation time ($t_s$), (v) hydraulic gradient ($\nabla \Psi$) is uniform over
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depth, (vi) saturated water front \( (z_s) \) advances parallel to the dike surface (pores have constant size along the infiltration depth), (vii) van Genuchten’s (1980) soil-water characteristic curve is used.

The infiltration water front \( (z_w) \) is calculated assuming that the parameter \( f \) defined by Weißmann (2003) and calibrated on finite element solver of Richard’s equation can be extended to unsaturated soil as a function of the saturated water front \( (z_s) \):

\[
Z_w - Z_s = \frac{Z_s}{f} \Rightarrow Z_w = \left(1 + \frac{1}{f}\right)Z_s \approx 15Z_s
\]

\[4.4\]

a) Weißmann’s model (Weißmann, 2003):

Saturated water front \( (z_s) \):

\[
t = \frac{\theta_s - \theta_r}{k_s} \left(\Delta \Psi_m + \bar{h}(t) + z_s(t)\right) - \ln \left(\frac{\Delta \Psi_m + \bar{h}(t)}{\Delta \Psi_m + \bar{h}(t) + z_s(t)}\right)
\]

Mean flow depth \( (\bar{h}) \) over the simulation time \( (t_s) \):

\[
\bar{h}(t_s) = \frac{\int_{t_0}^{t_s} h(t) dt}{t_s}
\]

Hydraulic gradient due to suction \( (\Delta \Psi_m) \):

\[
\Delta \Psi_m \equiv f\left[\Psi(\theta_s)\right]
\]

Model parameter: \( f = \frac{z_w}{z_w - z_s} \approx \frac{1}{14} \)

b) Wang Z.’s model (Wang, 2000a):

Saturated water front \( (z_s) \):

\[
z_s(t) = \sqrt{2\alpha k_s \int_{t_0}^{t} h(t) dt}; \quad \alpha = 4.5 \text{ empirical coefficient}
\]

Infiltration water front \( (z_w) \):

\[
z(t) = \sqrt{2\beta k_s \int_{t_0}^{t} h(t) dt}; \quad \beta \text{ empirical coefficient (eq. [4.5])}
\]

c) Wang Q.’s model (Wang et al., 2003):

Saturated water front \( (z_s) \): not strictly defined

Infiltration water front \( (z_w) \):

\[
t = \frac{\theta_s - \theta_r}{(1 + \alpha)k_s} \left(\ln(\beta z_w + 1)\right)
\]

Model parameters: \( \alpha = \frac{N_{bc}}{M} \); \( M = 2 + 3N_{bc} \)

\( \beta = \frac{M}{a \Psi_{sec}} \); \( a \approx 1 \) empirical coefficient (Wang et al., 2003)

Figure 4.4: Simplified infiltration models: saturated \( (z_s) \) and infiltration \( (z_w) \) water fronts

- Wang Z.’s model (Wang, 2000a)

The model calculates the saturated water front \( (z_s) \), similarly to Weißmann’s model solving Darcy’s law and assuming: (i) saturated soil, (ii) hydraulic gradient only function of the free surface flow depth \( (h) \), (iii) influence of unsaturated soil
on infiltration concentrated in the empirical parameter $\alpha$ (Figure 4.4b). Comparison of results with a finite element solver of Richard’s equation is used for validation.

The infiltration water front ($z_w$) is calculated assuming a functional relation similar to the saturated water front ($z_s$), but validation is still lacking (Wang, 2000a) and there are no precise indications on values of the empirical parameter ($\beta$). Therefore, the empirical parameter ($\beta$) is calculated by making the ratio between infiltration ($z_w$) and saturated ($z_s$) water fronts and making use of the parameter $f$ of the Weißmann’s model (Eq. [4.4]):

$$\frac{z_w}{z_s} = \frac{\beta}{\alpha k_s} \Rightarrow \beta = \alpha k_s \frac{z_w}{z_s} \approx 15\alpha k_s \tag{4.5}$$

- **Wang Q.’s model (Wang et al., 2003)**

It solves Richard’s equation for vertical infiltration and is validated against laboratory data, but not tested with waves (Tzimopoulos et al., 2005; Wang et al., 2003). Main assumptions are: (i) 1D vertical infiltration, (ii) Brooks and Corey’s (1964) soil-water characteristic curve is used, (iii) the model is derived from a previous solution proposed in the literature (Parlange, 1971) by using a Taylor’s series method (Figure 4.4c). It analytically calculates the infiltration water front ($z_w$).

The saturated water front ($z_s$) is not strictly defined because the volumetric water content ($\theta$) immediately decreases from the infiltration surface ($z = 0$).

**(ii) Volumetric water content**

After calculating the saturated ($z_s$) and infiltration ($z_w$) water fronts with one of the three selected models (Figure 4.4), the volumetric water content ($\theta$) is optionally derived in two ways:

- **Wang Q.’s model** (coefficient $\alpha$ defined in Figure 4.4c):

$$\theta(z) = \theta_r + \left(1 - \frac{z}{z_w}\right)^\alpha (\theta_s - \theta_r) \tag{4.6}$$

- Simply assuming a linear distribution between saturated ($z_s$) and infiltration ($z_w$) water fronts with the boundary values:

$$\theta(z_s) = \theta_s \text{ and } \theta(z_w) = \theta_i \tag{4.7}$$

**(iii) Suction pressure**

Suction pressure ($u$) is finally calculated from the volumetric water content ($\theta$) either with van Genuchten’s or Brooks and Corey’s soil-water characteristic curve (Table 4.4).
Combining the three models for saturated ($z_s$) and infiltration ($z_w$) water fronts (Figure 4.4) with the calculation of the volumetric water content ($\theta$) and of the suction pressure ($u$), six possibilities to solve water infiltration in the dike are proposed (Table 4.5).

<table>
<thead>
<tr>
<th></th>
<th>Saturated front $z_s$ [m]</th>
<th>Infiltration front $z_w$ [m]</th>
<th>Water content $\Theta(z)$ [m$^3$/m$^3$]</th>
<th>Suction pressure $u(z)$ [N/m$^2$]</th>
</tr>
</thead>
</table>

The influence of cracks at the inner slope on the infiltration is included as a function of their depth (d) and location (Figure 4.7). Calculation sections where cracks are located are assumed saturated up to crack depth (d).

Calculation of saturated ($z_s$) and infiltration ($z_w$) water fronts at the inner slope give nearly constant values, apart from sections with cracks and so does the volumetric water content ($\theta$). Wave overtopping thus produces almost uniform infiltration.

Comparison of saturated ($z_s$) and infiltration ($z_w$) water fronts calculated with the three selected models indicates very high differences in the results, due to (i) high uncertainties in the infiltration in the unsaturated zone (Weißmann’s model), (ii) partial validation of the models (Wang Z.’s model) and (iii) models not derived for infiltration due to wave overtopping (Wang Q.’s model):

- Weißmann’s model predicts both saturated ($z_s$) and infiltration ($z_w$) water fronts that are one/two orders of magnitude higher than those calculated with the other models (Figure 4.5), particularly for the infiltration water front ($z_w$);
- Wang Q.’s model does not predict the saturated water front ($z_s$) and it is less appropriate than the other models to be applied to the instability model of grass and clay;
- Model combinations 1b and 2a, b, c of Table 4.5 are recommended for use in the detailed model.

**Water infiltration** due to wave overtopping is not included in the available breach models (Chapter 2) and in the preliminary model (Chapter 3), but is included in the detailed model, although very simplified. A further significant improvement, not achieved in the present study, is represented by the application of a finite element infiltration model that directly solves Richard’s equation throughout the dike.
4.1.1.3 Uncertainties related to hydrodynamic aspects

The hydrodynamic module is partially based on the RANS-VOF Model COBRAS and partially on the same formulae and simple equations used in the preliminary model (Chapter 3). Input parameters of free surface flow are the same as for the preliminary model (Section 3.1.1.4). Input parameters of water infiltration are associated with high uncertainties. Typical values of coefficients of soil-water characteristic curve are measured from soil samples, but in some cases, available measures provide contrasting values (D’Eliso et al., 2006a). Moreover, the initial volumetric water content ($\theta_i$) is site-specific and time-dependent and is normally not measured, but assumed.

Model parameters include:
- Coefficients calibrated on laboratory tests for wave overtopping (Section 3.1.1.4);
- Empirical coefficients included in the simplified infiltration models like coefficients $\alpha$ and $\beta$ of Wang Z.’s model (Section 4.1.1.2).

4.1.2. Improvements of the morphodynamic module

The morphodynamic module includes (i) erosion and sediment transport and (ii) mass instability models from initial to the final breach.

The structure of the morphodynamic module is similar to that of the preliminary model. The whole process is also described in six phases (Figure 3.10), but here the clay cover failure is calculated and not assumed (Figure 4.6).
Breaching of sea dikes initiated by wave overtopping  

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<table>
<thead>
<tr>
<th>Phase</th>
<th>Description</th>
<th>Improvements</th>
</tr>
</thead>
</table>
| 1-2   | Phase 1: Grass erosion  
Phase 2: Local clay erosion  
**Improvements:** breach initiation with an initial scenario at the inner slope |
| 3a    | Phase 3a: Headcut erosion (scour erosion and headcut advance)  
**Improvements:** headcut advance is calculated with a discrete model, scour hole width is included |
| 3b    | Phase 3b: Headcut erosion of sand-clay scour (scour erosion and headcut advance) up to the total erosion or instability of the clay cover  
**Improvements:** neglected in the preliminary model |
| 3c    | Phase 3c: Cover failure and initial breach channel  
**Improvements:** grass and clay instability due to infiltration are neglected in preliminary model, the initial breach channel is calculated and not assumed |
| 4     | Phase 4: Crest shortening |
| 5     | Phase 5: Dike lowering |
| 6     | Phase 6: Full and final breach |

Figure 4.6: Improvements of the morphodynamic module in the detailed model (see also Figure 3.10 for the preliminary model)

Improvements are essentially related to the first three phases (grass and clay cover erosion and failure), which are very important for the whole process because (i) warning time \((t_w)\) may be defined as the time of cover failure \((t_{cf})\), (ii) no breach models that include the clay cover with grass are available and (iii) wave action is important up to the threshold time \((t_t)\), shortly after failure of the clay cover (Section 3.2.2). Improvements include:

- Breach initiation (Section 4.1.2.1);
- Mass instability and failure of the clay cover with grass (Section 4.1.2.2);
- Headcut erosion in the cohesive cover (Section 4.1.2.3);
- Headcut erosion in sand-clay scour (Section 4.1.2.4).

Some general assumptions of the preliminary model have been removed:

- Breach initiation (phase 1 of Figure 4.6) is detected according to an initial scenario approach instead of simply assuming a weak section at the inner slope;
- Headcut advance in the cohesive cover (phase 3a of Figure 4.6) is calculated using a discrete approach instead of an averaged continuous approach, thus explicitly including mass failure from the headcut face;
- Clay cover is not instantaneously removed as soon as the sand core is exposed to erosion, but the cover failure due to sliding or up-lift and the headcut erosion and advance of sand-clay scour hole are simulated (phase 3b-3c of Figure 4.6).

The morphodynamic module uses the outcomes of the hydrodynamic module (Section 4.1.1) as well as grass, clay and sand properties as inputs like in the preliminary model. The selected models calculate the time of breaching ($t_b$), the headcut height ($H_{hi}$), the breach height ($H_b$) and the breach width ($B_b$).

### 4.1.2.1 Breach initiation

Breach initiation at the inner slope is very complex, as it includes 3D stochastic mechanisms and depends on non-homogeneities of the loading and of the strength properties along the dike. Very few information are available on these issues and none of the existing breach models includes a breach initiation model. Field observations of real sea dikes (TAW, 2000) indicate typical appearances of the inner slope, that are candidates for incipient breach:

- Spots with damaged grass cover (Figure 4.7a);
- Cracks due to shrinkage in the clay cover (Figure 4.7b);
- Holes dig by animals in the clay cover (Figure 4.7c);
- Spots with strongly fractured soil due to weathering or sand-clay lenses (Figure 4.7d).

Breach initiation in the model is assumed to be a 2D process:

- Breach distribution along the dike line is not accounted for and the incipient breach is detected only at a representative cross-section;
- Incipient breach is located at a "weak section", identified by providing an initial scenario at the inner slope.

A qualitative description of the inner slope is given as an input in the model by defining type, geometry and severity of sections that are weaker than on average at the inner slope, i.e. weak sections, but it is not predicted (Figure 4.7). The combination of all weak sections provides the initial scenario that may lead to erosion and/or mass instability. There is a relation in the model, between type of weak section and grass/clay properties (Table 4.6).

The severity of weak sections provides indications on how big is the difference between local value of grass/clay properties and their average value along the inner slope. Four severity classes have been defined (Table 4.7).

The proposed breach initiation model automatically includes the simpler technique used in the preliminary model to detect the incipient breach location (Section 3.1.2.1). It is quasi process-oriented, but based on a given fixed scenario without including the stochastic distribution of non-homogeneities of the clay cover with grass. Sensitivity analysis is, particularly in this case, of great importance in order to understand the relative importance of each type of weak section and to define worst and best initial scenarios (Figure 5.1b).
1) Types of weak sections

a) Damaged grass
b) Cracks
c) Hole
d) Fractured clay, sand lens

2) Geometry and description of weak sections

<table>
<thead>
<tr>
<th>Type</th>
<th>Location</th>
<th>Length L [m]</th>
<th>Width W [m]</th>
<th>Depth d [m]</th>
<th>Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single crack</td>
<td>X</td>
<td>-</td>
<td>-</td>
<td>X</td>
<td>-</td>
</tr>
<tr>
<td>Hole</td>
<td>X</td>
<td>-</td>
<td>X</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>Damaged grass</td>
<td>X</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>Fractured soil</td>
<td>X</td>
<td>X</td>
<td>-</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Sand-clay lens</td>
<td>X</td>
<td>X</td>
<td>-</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Figure 4.7: Appearance and definition of weak sections (see Annex C for references)

Table 4.6: Relation between type of weak section and affected parameters and properties

<table>
<thead>
<tr>
<th>Type</th>
<th>Erosion</th>
<th>Mass instability</th>
<th>Affected parameters and properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single crack</td>
<td>-</td>
<td>X</td>
<td>Local infiltration fronts (z_s, z_w)</td>
</tr>
<tr>
<td>Hole</td>
<td>X</td>
<td>X</td>
<td>Local infiltration fronts (z_s, z_w) Grass cover factor (C_f)</td>
</tr>
<tr>
<td>Damaged grass</td>
<td>X</td>
<td>-</td>
<td>Grass cover factor (C_f)</td>
</tr>
<tr>
<td>Fractured soil</td>
<td>X</td>
<td>X</td>
<td>Saturated hydraulic conductivity (k_s,c) Grass root cohesion (c_p) Clay cohesion (c_c)</td>
</tr>
<tr>
<td>Sand-clay lens</td>
<td>-</td>
<td>X</td>
<td>Saturated hydraulic conductivity (k_s,c) Plasticity Index (I_p) Clay sediment size (D_{75,c}) Clay cohesion (c_c)</td>
</tr>
</tbody>
</table>
Table 4.7: Severity classes of weak sections and grass/clay properties (Sections 3.1.2.1, 3.1.2.2, 4.1.1.2 and 4.1.2.2)

<table>
<thead>
<tr>
<th>Severity classes</th>
<th>Description</th>
<th>Grass and clay properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>• locally removed grass</td>
<td>( C_{r,l} = 0; \ k_{u,l} = 10^{5} k_{u,l}; \ c_{p,l} = 0 )</td>
</tr>
<tr>
<td></td>
<td>• no soil strength</td>
<td>( c_{u,l} = 0; \ I_{r} = 0; \ D_{m,l} = 1.60D_{m,s} )</td>
</tr>
<tr>
<td>L</td>
<td>• low grass resistance</td>
<td>( C_{r,l} = 1/4C_{r}; \ k_{u,l} = 10^{5} k_{u,l}; \ c_{p,l} = 1/4c_{p,l} )</td>
</tr>
<tr>
<td></td>
<td>• low soil strength</td>
<td>( c_{u,l} = 1/4c_{u,l}; \ I_{r} = 1/4I_{r}; \ D_{m,l} = 1.40D_{m,s} )</td>
</tr>
<tr>
<td>M</td>
<td>• half grass resistance</td>
<td>( C_{r,l} = 2/4C_{r}; \ k_{u,l} = 10^{5} k_{u,l}; \ c_{p,l} = 2/4c_{p,l} )</td>
</tr>
<tr>
<td></td>
<td>• half soil strength</td>
<td>( c_{u,l} = 2/4c_{u,l}; \ I_{r} = 2/4I_{r}; \ D_{m,l} = 1.25D_{m,s} )</td>
</tr>
<tr>
<td>H</td>
<td>• 3/4 of grass resistance</td>
<td>( C_{r,l} = 3/4C_{r}; \ k_{u,l} = 10k_{u,l}; \ c_{p,l} = 3/4c_{p,l} )</td>
</tr>
<tr>
<td></td>
<td>• 3/4 soil strength</td>
<td>( c_{u,l} = 3/4c_{u,l}; \ I_{r} = 3/4I_{r}; \ D_{m,l} = 1.10D_{m,s} )</td>
</tr>
</tbody>
</table>

\( \_l = \) local values of material properties

4.1.2.2 Mass instability of the clay cover with grass

Mass instability of the grass and clay cover due to water infiltration at the inner slope may occur according to four failure mechanisms (Figure 4.8):

- Sliding of grass cover (turf set-off) at the turf base (\( Z = \Delta z_{g} \), Figure 4.8a);
- Sliding of clay cover at the saturated water front (\( Z = z_{s} \), Figure 4.8b);
- Sliding of clay cover at sand-clay interface (\( Z = S_{c} \), Figure 4.8c);
- Up-lift of clay cover at sand-clay interface (\( Z = S_{c} \), Figure 4.8d).

Figure 4.8: Mechanisms of mass instability of the clay cover with grass

Sliding of grass and clay cover represents a probable failure mode of saturated soils which have a higher weight (W) and associated higher mobilising force (\( F_{mo} \)) than unsaturated soils. Sliding of the clay cover at sand-clay interface, although not saturated, is also included.
Up-lift of the clay cover is a typical failure mode along discontinuities in the material such as a sand-clay interface ($Z = S_c$). It generally occurs after a long infiltration in the dike with low hydraulic gradients (Weißmann, 2003). Mass instability of the grass and clay cover is verified in the model by applying a conventional limit equilibrium approach (Figure 4.9). The influence of the grass cover on turf set-off is all included in the total (grass and clay) cohesion ($c_{tot}$).

Existing models (Wang, 2000b; Weißmann, 2003; Wu, T.H. et al., 1979), derived for slopes of infinite length and saturated cohesive soils, are extended in the detailed model to slopes of finite length and non-saturated cohesive soils with grass (Figure 4.9).

(i) Mobilising force ($F_{mo}$):
- **Weight of failing block** ($W$) considering submerged soil: $W = W_c + W_w$
  \[ W_c = (\rho_c - \rho_g)gZ_s \cos \beta \]
  \[ W_w = \rho_c g (Z-Z_{sm}) \ell \]
- **Pressure exerted by the flow** ($S$), upward directed (water infiltration lifts up the soil):
  \[ S = \rho g \frac{Z_s}{\ell} \]
  $S_c = S \cos \theta$; $S_s = S \sin \theta$; $\theta = 90^\circ - \arctan(J)$; Energy slope ($J$): $J = \frac{dh}{dx} = \frac{h_1 - h_2}{\ell}$
- **Active earth pressure** ($P_a$): $P_a = P_{a,s} + P_{a,u}$
  - Pressure of saturated soil: $P_{a,s} = 0.5K_s(\rho_s - \rho_g)gZ_{sm}$
  - Pressure of unsaturated soil: $P_{a,u} = 0.5K_s\rho_s g(Z - Z_{sm})$
  - Pressure of saturated on unsaturated soil: $P_{a,us} = K_a(\rho_s - \rho_g)gZ_{sm}(Z - Z_{sm})$
  Active earth pressure coefficient ($K_a$): $K_a = \tan^2 \left(45^\circ - \frac{\phi_c}{2}\right)$
- **Passive earth pressures** ($P_p$): $P_p = P_{p,s} + P_{p,u} + P_{p,us}$ (Conservative case: $P_p = 0$)
  - Pressure of saturated soil: $P_{p,s} = 0.5K_p(\rho_s - \rho_g)gZ_{sm}$
  - Pressure of unsaturated soil: $P_{p,u} = 0.5K_p\rho_s g(Z - Z_{sm})$
  - Pressure of saturated on unsaturated soil: $P_{p,us} = K_p(\rho_s - \rho_g)gZ_{sm}(Z - Z_{sm})$
  Passive earth pressure coefficient ($K_p$): $K_p = \tan^2 \left(45^\circ + \frac{\phi_c}{2}\right)$

Continued …
Chapter 4 Detailed model: development, implementation and tentative validation

(ii) Resisting force \((F_{re})\):

- **Shear strength for unsaturated soil \((R)\):** \(R = \tau + C \tan \phi_c\)
  
  Effective shear stress approach (Khalili & Khabbaz, 1999):
  \[
  \tau = c_w + \left[ (\sigma - u_z) + \chi (u_z - u_a) \right] \tan \phi_c; \quad \chi = \left( \frac{u_z - u_a}{u_b} \right)^{0.55} = \left( \frac{u_z}{u_b} \right)^{0.55}; \quad u_a = 0
  \]

  Cohesion \((C)\): \(C = c_w \ell\)

Normal resisting force \((N)\): \(N = N_s + N_u; \quad N_s = W_\perp - S_\perp; \quad N_u = -\int_a^b \left[ u_z + \chi (u_z - u_a) \right] \, d\ell\)

Figure 4.9: Mass instability of grass and clay cover: definition sketch and equations for turf set-off, clay cover sliding and up-lift

The block of failing soil slides along the inner slope when the mobilising force \((F_{mo})\), i.e. shear stress, exceeds the resisting force \((F_{re})\) i.e. shear strength, giving a factor of safety (FOS) lower than 1:

\[
\text{FOS} = \frac{F_{re}}{F_{mo}} < 1 \quad [4.8]
\]

1) **Turf set-off and sliding of the clay cover** (Figure 4.8a, b, c):

- Mobilising force \((F_{mo})\): \(F_{mo} = W_// + S_// + P_a\) \quad [4.9]
- Resisting force \((F_{re})\): \(F_{re} = R + P_p\) \quad [4.10]

2) **Up-lift of the clay cover** (Figure 4.8d):

- Mobilising force \((F_{mo})\): \(F_{mo} = W_\perp\) \quad [4.11]
- Resisting force \((F_{re})\): \(F_{re} = S_\perp\) \quad [4.12]

The **total cohesion** \(c_{tot}\) (Figure 4.9) is the sum of the cohesion of clay \((c_c)\) and the cohesion due to grass roots \((c_g)\), which decreases over depth \(z\):

\[
c_{tot} = c_c + c_g \quad [4.13]
\]

The **grass root cohesion** \((c_g)\) to be included in the turf set-off model through the total cohesion \((c_{tot})\) is calculated at the turf base \((c_g = c_g(\Delta z_g))\), where the sliding surface is located. Several models (Gray & Ohashi, 1983; Michalowski & Zhao, 1996; Wu, T.H. et al., 1979) which have been further reviewed (Cazzuffi & Crippa, 2005; Coppin & Richards, 1990; Styczen & Morgan, 1995; Wang, 2000b; Young, 2005) are available to calculate the grass root cohesion \((c_g)\), as a particular case of fibre reinforced soils where the root network acts as a reinforcement. Among them, the model of Wu et al. (1979) is the most quoted: grass root cohesion \((c_g)\) in cohesive soils is empirically calculated as a function of the
average tensile strength of the grass root ($\sigma_g$), the angle of shear rotation ($\theta$) and the angle of friction of clay ($\phi_c$):

$$c_g = \sigma_g \left( \cos \theta \tan \phi_c + \sin \theta \right) \approx 1.2\sigma_g \quad \text{for} \quad \theta \in \left[48^\circ, 72^\circ\right]$$  \[4.14\]

The average tensile strength of the grass root decreases over depth ($z$) as a function of the root area ratio (RAR), which is defined as the ratio between the root area ($A_R$) and the soil area ($A$):

$$\sigma_g(z) = \sigma_g(z=0) \cdot \text{RAR}(z); \quad \text{RAR}(z) = \frac{A_R(z)}{A}$$  \[4.15\]

Typical values of the average tensile strength of the grass root at the soil surface ($\sigma_g(z=0)$) are given in Cazzuffi & Crippa (2005) as a function of the type of grass. The root area ratio (RAR) decreases over depth ($z$) according to an exponential law (Sprangers, 1999; Stanczak et al., 2006). From laboratory tests on grass, Stanczak et al. (2006) found:

$$\text{RAR}(z) = A \cdot D^{100z-2}; \quad A = 1.58 \text{ and } D = 0.75$$  \[4.16\]

Empirical coefficients $A$ and $D$ are function of the type and quality of grass, but a detailed investigation on their values is still not available and specific measures are recommended. If the grass root cohesion ($c_g$) cannot be calculated with eqs. [4.14]-[4.16], because specific measures are not available, it may be assumed in the range $1.5÷5.5$ kN/m$^2$ (Hewlett et al., 1987; Stanczak et al., 2006; Wu, T.H. et al., 1979), depending on grass quality ($1.5$ kN/m$^2$ for very low grass quality and $5.5$ kN/m$^2$ for high grass quality).

Parametric studies on the influence of geometry and material parameters on sliding (eqs. [4.8]-[4.10]), which represents a more likely failure mode than uplift, indicate that:

- The inner slope (1:n) mostly compromises stability (Figure 4.10a), thus confirming the results of previous studies (Kortenhaus, 2003): formation of an initial breach channel is rather caused by mass instability for very steep slopes ($n = 2÷3$) and by gradual erosion for gentle slopes ($n \geq 3$);
- The saturated water front ($z_s$) also represents a significant influencing parameter (Figure 4.10b): the deeper the saturated front ($z_s$) is (long wave storm and high water infiltration), the more affected the stability will be;
- The length of the failing block of soil ($l$) doesn’t influence the instability, if greater than about 30 m.

Mass instability of grass and clay cover due to water infiltration is not included in the breach models that are available in the literature (Chapter 2) and in
the preliminary model developed in Chapter 3, but is included in the detailed model, although applying a limit equilibrium approach. Further significant improvements, that are not included in this study, are the application of a finite element model (Chok et al., 2004) and the analysis of the turf set-off as a progressive rather than a sudden process (Young, 2005).

Figure 4.10: Influence of inner slope (n) and saturated water front (zs) on sliding on the factor of safety (FOS = Fren/Freo)

4.1.2.3 Headcut erosion in the cohesive cover

Headcut erosion initiates gradually as an evolution of the local clay erosion (phase 2 of Figure 4.6) and is the result of several mechanisms (Section 2.2.3), but is normally dominated by scour erosion and headcut advance (phase 3a of Figure 4.6), as simply included in the preliminary model (Section 3.1.2.2):

a) Headcut initiation (phase 3a)
Generally, headcut initiation is conventionally defined, rather than modelled. The preliminary model also makes use of a conventional and not process-oriented criterion (eq. [3.25]). In fact, a higher averaged flow discharge (qm) yields a larger initial headcut height (Hh,i), a higher time of headcut initiation (tH) and of cover failure tcf (Section 3.1.2.2). Laboratory tests (Bennett & Casalì, 2001) show however that a higher discharge (q) and a higher initial headcut height (HH,i) result in a lower time of headcut initiation (tH).

Defining a new criterion based on the bottom shear stress (τb) or the Froude number (Fr) at the headcut base produces similar results. A significant step toward the understanding of the headcut initiation process may be the use of a hydrodynamic model at the inner slope that includes the flow turbulence, in order to define an initiation criterion as a function of the turbulence at the headcut base.
b) Scour erosion (phase 3a)

The vertical erosion (dz) is in the model still proportional to the excess effective bottom shear stress (τ_{0,eff} - τ_{0,cr}) and is limited by an equilibrium scour (S_{eq}). Longitudinal (dx) and lateral (db) erosion is also included in the model.

The vertical erosion depth (dz) at the headcut base is calculated with eq. [3.21], like in the preliminary model. The effective bottom shear stress (τ_{0,e}) is the bottom shear stress applied at the headcut base and is a function of the flow in the impinging jet region (Rajararatnam & Muralidhar, 1968). It is calculated with eq. [3.29], where the effective bottom shear stress (τ_{0,e1}) is assumed equal to the value at the jet impact point (τ_{0,jet}), neglecting the influence of the position of the jet entry point X_P (Stein et al., 1993), see Figure 4.11:

\[
\begin{align*}
\tau_{0,e1} &= \tau_{0,jet} = C_{d} \rho w v_{0}^{2} \quad \text{if} \quad J_{jet} \leq J_{jet,P} \\
\tau_{0,e1} &= \tau_{0,jet} = C_{d} \rho w v_{0}^{2} \frac{h_{B}}{J} \quad \text{if} \quad J_{jet} > J_{jet,P}
\end{align*}
\]

The jet diffusion coefficient (C_d) is equal to 2.60 (Stein et al., 1993).

![Diagram](image-url)

**Flow velocity** (v_i) and **depth** (h_i) in the accelerated flow region:

\[ v_i = \frac{Fr}{Fr} + 0.4 \sqrt{g} \quad h_i = \frac{q}{v_i} \quad Fr = \frac{v}{\sqrt{gh}} \]

**Flow velocity** (v_0) and **depth** (h_0) at the entry point of the jet (energy conservation):

\[ v_0 = \sqrt{v_i^2 + 2g(H_{ni} + h_i/2 - h_i)} \quad h_0 = \frac{q}{v_0} \]

Backwater flow depth in impinging region (h_B): \ h_B \approx h

**Friction coefficient at the jet impact point**, Blasius formula (C): \[ C = 0.22 \left( \frac{h_{ni} v_{ni}}{v} \right)^{0.25} \]

Jet impact point (X_P): \[ X_P = v_i \sqrt{\frac{2(H_{ni} + h_i/2 - h_i)}{g}} \]

Figure 4.11: Flow in the accelerated and impinging jet regions: definition sketch and equations according to Rajaratnam & Muralidhar (1968) and Zhu (2006)
The **scour hole depth** \((S)\), which is the cumulated vertical erosion \((\Sigma dz)\), is limited by an equilibrium value \((S_{eq})\). At equilibrium, headcut advances seaward without deepening. The **equilibrium scour hole depth** \((S_{eq})\) is the scour hole depth \((S)\) related to a shear stress at the jet impact point \((\tau_{0,jet})\) equal to the critical shear stress \((\tau_{0,cr})\). It is calculated by using eq. [4.17] and neglecting the backwater flow depth \(h_B\) (Stein et al., 1993), see Figure 4.11:

\[
S_{eq} = J_{jet} \sin(\chi_{jet}) - h_B \approx J_{jet} \sin(\chi_{jet}) = \frac{C_d^2 \rho_v v_0^2 h_0}{\tau_{0,cr}} \sin(\chi_{jet})
\]  

Longitudinal \((dx)\) and lateral \((db)\) erosion can be strictly calculated with 3D flow and 3D headcut erosion models. In fact, the impinging jet at the impact point, turns mostly upstream and reaches the headcut base, where it moves up to the vertical face and turns downstream, partially expanding in the transverse direction (Frenette & Pestov, 2005). The resulting shear stress field is 3D. Since available headcut erosion models only calculate vertical erosion \(dz\) (Section 2.2.3), assumptions are required (Figure 4.11):

(i) **Longitudinal** \((dx)\) and **vertical** \((dz)\) erosion are assumed to have the same order of magnitude (Zhu, 2006):

\[
\frac{dz/dt}{dx/dt} = 1
\]

(ii) **Lateral erosion** \((db)\) is function of **vertical erosion** \((dz)\) through a proportionality coefficient \((c_{db/dz,H})\), given as input parameter. Based on observations of the ratio between the headcut height \((H_H)\) and width \(B_H\) (MacDonald & Landgridge-Monopolis, 1984) and on existing models for headcut erosion of cohesive soils (Robinson & Hanson, 1994; Zhu, 2006), a range of possible values for the proportionality coefficient \((c_{db/dz,H})\) is suggested:

\[
c_{db/dz,H} = \frac{db/dt}{dz/dt}; \quad c_{db/dz,H} = 0.5 \div 1.5
\]

The **time of cover erosion** \(t_{ce}\) (end of phase 3a of Figure 4.6) is defined as the scour hole depth \((S)\) equals the minimum between the equilibrium scour hole depth \((S_{eq})\) and the cover layer thickness \((S_c)\), when sand is exposed to erosion.

**c) Headcut advance (phase 3a)**

Two **modes of headcut advance** have been observed during laboratory tests:

(i) Rotating headcut that migrates altering their shape;

(ii) Stepped headcut that have constant vertical shape.
Field observations normally indicate that stepped headcut is more likely to occur. Separation between the two modes depends on the ratio between time-scale of erosion at the headcut brink and base (Stein & Julien, 1993). Stepped headcut, corresponding to dominating erosion at the base, develops with low undisturbed flow depths \( h \), high Froude numbers \( F_r \) and high initial headcut heights \( H_{hi} \), as during dike breaching. Application of the model of Stein & Julien (1993) to wave overtopping flows, also indicate that mode of advance is always stepped (D’Eliso et al., 2006a).

The headcut advance \( dX \) of a stepped headcut is calculated with a discrete model of mass instability from the vertical headcut face by applying a conventional limit equilibrium approach, assuming a vertical failure surface defined by the scour erosion (Figure 4.12a), a width of the failing block of soil \( (B_{hi}) \) equal to width of the headcut scour at the headcut base (Figure 4.12b) and three possible modes of failure (shear, overturning and bending).

\[
\begin{align*}
S_c &= \text{Thickness of clay cover} \\
W_c &= \rho_c g (H_{hi} - (dz_c)) (dx_c) B_{hi} \\
W_w &= \rho_w g (dx_c) B_{hi} \\
F_w &= \rho_w u_c H_{hi} (dx_c) B_{hi} \\
&= \tau \rho = g \frac{h_{hi} - (dz_c)}{2} B_{hi} \\
P_h &= \begin{cases} 
\rho_c g \left( \frac{(h_{hi} - (dz_c))^2}{2} \right) B_{hi} & \text{if } h_{hi} > (dz_c) \\
0 & \text{if } h_{hi} \leq (dz_c)
\end{cases} \\
P_v &= \begin{cases} 
\rho_w g (h_{hi} - (dz_c)) (dx_c) B_{hi} & \text{if } h_{hi} > (dz_c) \\
0 & \text{if } h_{hi} \leq (dz_c)
\end{cases} \\
R &= C + N \tan(\phi_c) \\
C &= C_i + C_i \\
C_i &= \rho_c g (H_{hi} - (dz_c)) B_{hi} \\
C_l &= 2 \rho_c g (H_{hi} - (dz_c)) (dx_c) \\
N &= F_x
\end{align*}
\]

Figure 4.12: Definition sketch of discrete headcut advance model for cohesive soils
The suction pressure ($u$) is neglected because at instability, the failure surface is saturated by infiltration due to crack formation.

Shear failure is typical for non-cohesive soils, while bending and overturning failures are typical for cohesive soils (Hassan, 2002). In the model, the three modes of failure are optionally selected, but the overturning mode is suggested as the most likely (Zhu, 2006):

- **Shear failure**

  The soil block falls in the scour hole when the mobilising force ($F_{mo}$), i.e. shear stress, exceeds the resisting force ($F_{re}$), i.e. shear strength, giving a factor of safety (FOS) lower than 1 (Figure 4.12):

  $FOS = \frac{F_{re}}{F_{mo}} < 1$  
  \[ [4.21] \]

  - Mobilising force ($F_{mo}$):  
    \[ F_{mo} = W - P_v - P_h \tan(\phi_c) \]  
    \[ [4.22] \]
  - Resisting force ($F_{re}$):  
    \[ F_{re} = R \]  
    \[ [4.23] \]

- **Bending failure**

  A tension crack appears across the failure surface, the soil block starts rotating around the headcut base, i.e. axis O-O (Figure 4.12b) and falls in the scour hole, as tensile stress ($\sigma$) exceeds tensile strength of clay ($\sigma_c$), giving a factor of safety (FOS) lower than 1 (Figure 4.12):

  $FOS = \frac{\sigma_c}{\sigma} < 1$  
  \[ [4.24] \]

  - Tensile stress ($\sigma$):
    \[ \sigma = \frac{(F_w - P_h)}{(H_H - \Delta z)B_h} - \frac{C_1}{2(H_H - \Delta z)\Delta h} + \frac{M_0}{(H_H - \Delta z)^2} \]  
    \[ [4.25] \]
    \[ M_0 = W_s e_{w_s} + W_c e_{w_c} + F_w e_{f_w} - C_1 e_{c_1} - C_1 e_{c_1} - P_h e_{p_h} - P_r e_{p_r} \]  
    \[ [4.26] \]

  Lever arms ($e_i$) of the corresponding forces are given in Figure 4.12.

  - Tensile strength of clay ($\sigma_c$): function of soil and normally estimated with compression tests on soil samples and included in the range 2000-3200 N/m$^3$ (Munkholm et al., 2002).

- **Overturning failure**

  It is conceptually similar to bending failure, but a tension crack appears and leads to failure soon as the mobilising moment ($M_{mo}$) becomes higher than the resisting moment ($M_{re}$) around the headcut base, i.e. axis O-O (Figure 4.12b):

  $FOS = \frac{M_{re}}{M_{mo}} < 1$  
  \[ [4.27] \]
Breaching of sea dikes initiated by wave overtopping

C. D’Eliso

- Mobilising moment (M_{mo}): \[ M_{mo} = W_s e_{w_s} + W_w e_{w_w} + F_w e_{F_w} \]  
- Resisting moment (M_{re}): \[ M_{re} = C_t e_{C_t} + C_i e_{C_i} + P_h e_{P_h} + P_s e_{P_s} \]

Comparison of the three aforementioned modes of failure related to stepped headcut advance indicates different time intervals between successive failures (\( \Delta t_{Hf} \)) and different time of cover erosion (\( t_{ce} \)). In particular, application to the prototype dike defined in Section 3.2 indicates that the overturning mode is associated with the minimum time interval (\( \Delta t_{Hf} \)) and the minimum time of cover erosion (\( t_{ce} \)), followed by shear and bending (Figure 4.13). In general, the higher the time interval (\( \Delta t_{Hf} \)) is, the higher the time of cover erosion (\( t_{ce} \)) and the lower the cumulated headcut advance (\( \Sigma dX \)) will be.

![Figure 4.13: Comparison of modes of failure in stepped headcut advance](image)

Improvements of headcut erosion and advance simulation in the present model include the calculation of erosion depths based on the flow in the impinging jet region and a discrete instead of a continuous averaged model for the headcut advance. A further improvement, not achieved in the present study, includes a more process-oriented 3D estimate of the shear stress pattern and erosion depths at the headcut base.

4.1.2.4 Headcut erosion in the sand-clay scour hole

Headcut erosion in sand-clay scour (phase 3b of Figure 4.6) is induced by scour erosion of sand and advance seaward of the clay cover (Geisenhainer & Kortenhaus, 2006), as in phase 3a of Figure 4.6, up to the complete failure of the clay cover (phase 3c of Figure 4.6).
a) Scour erosion (phase 3b)

The vertical erosion depth \(dz\) is calculated by using a sediment transport model that includes non-equilibrium sediment transport conditions (Nakagawa & Tsujimoto, 1980). The model accounts for both mean bottom shear stress \(\tau_{0m}\) and fluctuating lift forces \(L_P\) due to high flow turbulence in the impinging jet region as driving forces (Jia et al., 2001). The model is based on the calculation of the sediment transport discharge \(Q_sT\) as a difference between the volume of pick-up sediments \(V_p\) and the volume of deposited sediments \(V_d\) (Table 4.8). More details are given in D’Eliso et al. (2006a), Jia et al. (2001) and Nakagawa & Tsujimoto (1980).

Table 4.8: Sediment transport model adopted for scour erosion of sand (Jia et al., 2001)

<table>
<thead>
<tr>
<th>Mean sediment pick up rate (p_{sm}): (p_{sm} = \frac{p_{sm}}{\sqrt{2\pi D_{50,s}}} = \int_{-\infty}^{\infty} p_{sm} \psi(\eta) d\eta); (\eta = \frac{L_P}{\sigma_{L_P}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Sediment pick-up rate (p_s) is function of the fluctuating lift force (L_P), its standard deviation (\sigma_{L_P}), sediment properties, mean bottom shear stress (\tau_{0m}) calculated as in phase 3a of Figure 4.6 (Section 4.1.2.3) and empirical coefficients (Nakagawa &amp; Tsujimoto, 1980). The fluctuating lift force (L_P) and its standard deviation (\sigma_{L_P}) are functions of the flow at entry point and the turbulent kinetic energy relative to the value at jet impact point.</td>
</tr>
<tr>
<td>- Normal distribution of fluctuating lift forces (\Psi): (\psi(\eta) = \frac{1}{\sqrt{2\pi}} \exp \left(\frac{-\eta^2}{2}\right))</td>
</tr>
<tr>
<td>Exponential distribution of sediment step length (\xi): (f_\xi(\xi) = \frac{1}{\Lambda} \exp \left(-\frac{\xi}{\Lambda}\right))</td>
</tr>
<tr>
<td>- Sediment step length (\xi)</td>
</tr>
<tr>
<td>- Mean sediment step length (\Lambda): (\Lambda = f(D_{50,s}) = 2000 + 3000D_{50,s}) D’Eliso et al (2006a)</td>
</tr>
<tr>
<td>Pick-up volume (V_p) at section i in the scour: (V_p(i) = \frac{A_1D_{50,s}p_{sm}(i)\Delta x(i)}{A_2})</td>
</tr>
<tr>
<td>Deposited volume (V_d) at section i in the scour: (V_d = \frac{A_1D_{50,s}}{A_2} \sum_{i=1}^{l} p_{sm}(l)\Delta x(l) \int f_\xi(\xi) d\xi)</td>
</tr>
</tbody>
</table>

The vertical erosion depth \(dz\) is finally calculated applying 1D sediment continuity equation (Exner eq.):

\[
\frac{dz(i)}{dx} = \frac{A_1A_2}{A_3} \left(V_p(i) - V_d(i)\right) \frac{dt}{dx}; \quad A_1 = 1; \quad A_2 = \frac{\pi}{4}; \quad A_3 = \frac{\pi}{6}
\]

Where \(A_1\), \(A_2\) and \(A_3\) are one-, two- and three-dimensional geometrical coefficients of sand particles.

The Longitudinal \((dx)\) and lateral \((db)\) erosion are calculated with eqs. [4.19]-[4.20], which may be assumed still valid (Hassan et al., 1999; Kraus & Hayashi, 2005).
Sediment particles move from the jet impact point, both seaward and landward in the upstream and downstream roller (Figure 4.11). The volume of deposited sediments ($V_d$) is in fact minimum at the jet impact point and increases both seaward and landward (Figure 4.14). Due to limitations of the hydrodynamic module at the inner slope (Section 4.1.1.1), the turbulence kinetic energy and the averaged bottom shear stress ($\tau_{0m}$) are assumed constant in the scour, resulting in a constant sediment pick up rate ($p_s$) and a constant volume of pick-up sediments $V_p$ (Figure 4.14).

![Figure 4.14: Sediment pick-up ($V_p$) and deposited ($V_d$) volume in sand scour calculated by the model in Table 4.8](image)

**b) Headcut advance (phase 3b)**
The headcut advance ($dX$) is calculated with a discrete 2D model assuming that the failing block of soil behaves like a cantilever (Jia et al., 2002) and an overturning failure (Section 4.1.2.3 and Figure 4.15).

![Figure 4.15: Definition sketch of discrete headcut advance model in sand-clay scour](image)
Mobilising and resisting moments are calculated at the headcut base around the point O (Figure 4.15). The failure occurs as the mobilising moment \( M_{mo} \) becomes higher than the resisting moment \( M_{re} \):

\[
\text{FOS} = \frac{M_m}{M_{mo}} < 1
\]

- Mobilising moment \( M_{mo} \):
  \[ M_{mo} = W_e e_{w_e} + W_s e_{w_s} + F_e e_{f_e} \]  
  \[ [4.32] \]

- Resisting moment \( M_{re} \) is the moment of a cantilever:
  \[ M_{re} = \frac{1}{6} \sigma c \frac{S^2}{2} \]  
  \[ [4.33] \]

c) Clay cover failure (phase 3c)
The time of cover failure \( t_{cf} \) is defined at the end of phase 3b of Figure 4.6, either when the clay cover slides or lifts-up (Section 4.1.2.2) or when it is eroded up to the dike crest by the headcut advance seaward, resulting in an initial breach channel of rectangular cross-section at the inner slope (Phase 3c of Figure 4.6). The initial breach channel height \( H_{b,0} \) is assumed equal to cover layer thickness \( S_C \). The initial breach channel width \( B_{b,0} \) is equal to the final width of the sand scour \( B_{hi} \):

\[
B_{b,0} = B_{hi}
\]  
  \[ [4.34] \]

4.1.2.5 Uncertainties related to material properties

The morphodynamic module requires several input parameters (grass, clay and sand parameters), which are often highly uncertain, like in the preliminary model. New input parameters, influenced by the weathering of grass and clay, are grass root cohesion \( c_g \) and tensile strength of clay \( \sigma_c \). Definition of initial scenario and severity of weak sections at the inner slope for breach initiation is also a function of the weathering of grass and clay, the dike maintenance and the natural variability of materials.

A new model parameter has been introduced in the detailed model, namely the ratio between lateral and vertical erosion in the scour at the headcut base \( (c_{db/dz,H}) \): suggested values are based on past experiences and assumptions made in existing models, but due to the importance of this parameter, the associated uncertainties have to be considered in the uncertainty analyses (Chapter 5).

4.1.3. Limitations of the mathematical formulation

Seven major limitations of the mathematical formulation in the detailed model are worth to be mentioned:

(i) The calculation of the free surface flow and the water infiltration is not coupled;
(ii) Free surface flow model at the inner slope doesn’t include a turbulence model for erosion calculation;
(iii) Water infiltration in the dike due to high mean water level and rainfall is not included and water infiltration solely due to wave overtopping and combined flow is calculated using simplified models (Section 4.1.1.2);
(iv) Breach initiation doesn’t account for the stochastic nature of the location and type of weak sections along the inner slope, but is considered as purely deterministic;
(v) Turf set-off is not treated as a progressive mechanism, but is calculated using a constant grass root cohesion ($c_g$) over time;
(vi) Scour at the headcut base is not fully described, but only erosion at the headcut base is included;
(vii) 3D shear stress pattern at the headcut base is not really calculated, but only horizontal shear stress ($\tau_{0,\text{jet}}$) is included.

4.2. Overall model implementation

The detailed model is based on equations and models selected and described in Section 4.1 (Table 4.9). The same prototype sea dike defined for the preliminary model (Figure 3.17) is used for model application (Section 4.2.2) and for uncertainty analyses (Chapter 5). A complete list of input parameters is given in Annex B.

4.2.1. Modules of the detailed model

Like the preliminary model, the detailed model is also the result of a set of modules:

- **Input module**: reads the input parameters, defines waves, material properties and calculation dike sections (see also Section 3.2.1);
- **Hydrodynamic module**: solves both wave overtopping, overflow or combined flow equations and water infiltration in the dike (Sections 3.1.1 and 4.1.1);
- **Morphodynamic module**: solves erosion and instability models for grass, clay and sand, according to the six breaching phases described in Figure 4.6, introducing new processes and model improvements (Sections 3.1.2 and 4.1.2);
- **Output module**: saves model outcomes (see also Section 3.2.1).

For each phase, hydrodynamic and morphodynamic modules are iteratively applied until conditions for the successive phase are verified (Figure 4.16). Phases 4-6, relative to sand erosion have the same structure as in the preliminary model (Figure 3.18).
### Table 4.9: Models and equations adopted in the detailed model

<table>
<thead>
<tr>
<th>Phases</th>
<th>Models and equations</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave overtopping</td>
<td>RANS-VOF Model COBRAS Schüttrumpf formulae at the inner slope</td>
<td>Figure 3.4, Table 4.1 eqs. [3.1]-[3.4] eq. [3.11]</td>
</tr>
<tr>
<td>Combined flow</td>
<td>Steady non-uniform free surface flow equations Weir - Contracted flow formulae</td>
<td>Figure 3.7, Figure 3.8 eqs. [3.5]-[3.14]</td>
</tr>
<tr>
<td>Overflow</td>
<td>Simplified solutions of Richard’s equation</td>
<td>Figure 4.4, Table 4.5 eqs. [4.4]-[4.7]</td>
</tr>
<tr>
<td>Turf set-off</td>
<td>Limit equilibrium approach</td>
<td>Figure 4.9 eqs. [4.8]-[4.10]</td>
</tr>
<tr>
<td>Cover sliding</td>
<td>Limit equilibrium approach</td>
<td>Figure 4.9 eqs. [4.8], [4.11]-[4.12]</td>
</tr>
<tr>
<td>Cover up-lift</td>
<td>Initial scenario approach</td>
<td>Figure 4.7, Table 4.7</td>
</tr>
<tr>
<td>Phase 0</td>
<td>Grass erosion</td>
<td>Effective bottom shear stress eqs. [3.15]-[3.20]</td>
</tr>
<tr>
<td>Phase 1</td>
<td>Grass erosion</td>
<td>Excess bottom shear stress eqs. [3.21]-[3.24]</td>
</tr>
<tr>
<td>Phase 2</td>
<td>Headcut erosion in cohesive cover</td>
<td>Excess bottom shear stress Discrete advance model Figure 4.11, Figure 4.12 eqs. [3.29], [4.17] eqs. [4.19]-[4.29]</td>
</tr>
<tr>
<td>Phase 3a</td>
<td>Headcut erosion in sand-clay scour</td>
<td>Non-equilibrium sediment transport and sediment continuity equation Discrete advance model Figure 4.15, Table 4.8 eqs. [4.17], [4.19]-[4.20] eqs. [4.27]-[4.28] eqs. [4.30]-[4.33]</td>
</tr>
<tr>
<td>Phase 3b</td>
<td>Cover failure</td>
<td>Initial breach from final scour eq. [4.34]</td>
</tr>
<tr>
<td>Phase 4</td>
<td>Sand erosion-crest</td>
<td>Sediment transport formulae: Bagnold-Visser or Yang</td>
</tr>
<tr>
<td>Phase 5a and 5b</td>
<td>Breach morphology: Exner equation parametric relations</td>
<td>eqs. [3.36]-[3.49] Table 3.5, Table 3.6</td>
</tr>
<tr>
<td>Phase 6a and 6b</td>
<td>Sand erosion Full breach</td>
<td>Breach slopes instability: limit equilibrium approach (shear) Figure 3.15, Figure 3.16</td>
</tr>
<tr>
<td>Loading cases (Section 4.1.1)</td>
<td>Loading cases (Section 4.1.1)</td>
<td></td>
</tr>
</tbody>
</table>

### 4.2.2. Computational aspects, results and discussion

Definition of calculation sections along the dike profile and the related grid spacing (Δx) and time step (Δt) are similar to the preliminary model (Section 3.2.2.1). The selection modus of horizontal (Δx) and vertical (Δz) grid spacing and time step (Δt) of RANS-VOF Model COBRAS is given in (Liu & Lin, 1997). Sand-clay scour model (Section 4.1.2.4) requires the definition of local calculation sections in the scour and the related grid spacing (Δx), without any restrictions due to numerical instabilities.
Application of the model system to the prototype dike defined in Section 3.2 (Figure 3.17) for wave overtopping of irregular waves provides typical outcomes from the detailed model and indicates the main differences between the preliminary and the detailed model (Table 4.10).

The comparative analysis of the outcomes from both models shows the following with respect to the time associated to breaching, the erosion in clay and sand scour, the headcut advance, the breach outflow hydrograph and the breach widening:

(i) **Time associated to breaching** (phases 1-6 of Figure 4.6)

The *time of grass failure* ($t_{fg}$) is 47% lower than in the preliminary model, due to the different technique to assess breach initiation. In fact, in the preliminary
model, as initiation mechanism, only a weak section with damaged grass may be accounted for, while in the detailed model an initial scenario that includes several type of weak sections is defined (Section 4.1.2.1). The time of cover erosion (tce) is 22% higher than in the preliminary model. This difference increases up to 57% if the time associated only to clay and headcut erosion in the cohesive cover (tce-tgf, phases 2 and 3a) are compared. This is a consequence of the two different implemented headcut models (Figure 4.18). The time of cover failure (tcf) and consequently the warning time (tw) are 23% higher than in the preliminary model. This difference increases up to 58% considering the time associated only to clay and headcut erosion (tcf-tgf, phases 2 and 3). The prediction of the time of cover failure (tcf) is more accurate in the detailed model because the headcut erosion of sand-clay scour is also simulated, although it is very fast as compared to the erosion and failure of the cover layer (tcf-tce ≈ 2% tcf).

The relative influence of core and clay cover erosion on the time of breaching (tb) is the same as observed in the preliminary model (tb-tcf ≈ 5% tcf), see Section 3.2.2. The time of breaching (tb) is higher in the detailed model (22%) as a consequence of the higher time of cover failure (tcf).

| Table 4.10 Application of the Detailed Model (DM) to the prototype dike: outcomes and comparison with the Preliminary Model (PM) |
|---------------------------------|----------------|----------------|----------------|----------------|----------------|
| **Outcome**                      | **Symbol**    | **SI unit**    | **PM**         | **DM**         | **ε [%]**     |
| Unit discharge (t < t<sub>t</sub>) | q<sub>m</sub> = V<sub>0,i</sub>/B<sub>0i</sub>t<sub>i</sub> | m<sup>3</sup>/sm | 0.033 | 0.033 | - |
| Peak outflow discharge           | Q<sub>b,p</sub> | m<sup>3</sup>/s | 1192 | 933 | 28 |
| Total released water volume      | V<sub>0,0</sub> | m<sup>3</sup> | 650417 | 650022 | ≈ 0 |
| Time of grass failure            | t<sub>gf</sub> | hr | 3.63 | 2.47 | 47 |
| Time of headcut initiation       | t<sub>H</sub> | hr | 4.39 | 3.02 | 45 |
| Time of cover erosion            | t<sub>ce</sub> | hr | 5.71 | 7.32 | 22 |
| Time of cover failure            | t<sub>cf</sub> | hr | 5.71 | 7.44 | 23 |
| Time of dike failure             | t<sub>df</sub> | hr | 5.84 | 7.53 | 22 |
| Time of breaching                | t<sub>b</sub> | hr | 6.03 | 7.78 | 22 |
| Erosion rate (phase 2)           | dz/dt         | mm/s | 0.07 | 0.11 | 36 |
| Erosion rate (phase 3)           | dz/dt         | mm/s | 0.08 | 0.06 | 33 |
| Headcut height growth rate       | dH/dt (phase 3a) | mm/s | 0.13 | 0.07 | 86 |
| Headcut advance rate             | dX/dt (phase 3a) | mm/s | 0.34 | 0.05 | 580 |
| Initial breach channel width     | B<sub>i</sub> = B<sub>i</sub> | m | 3.00 | 5.39 | 44 |
| Final breach width               | B<sub<f</sub> | m | 58.69 | 45.22 | 30 |
| Wave overtopping:                |               |               | V<sub>0,i</sub> Total released water volume at t = t<sub>f</sub> | Difference between PM and DM: ε = 100|DM − PM|DM |
| H<sub>5</sub> = 3.00 m, T<sub>p</sub> = 10.0 s, SWL = 6.50 m |

(ii) Scour erosion in clay and sand (phases 2-3b of Figure 4.6)
During clay (phase 2) and headcut erosion (phases 3a-3b), there are three regions with different vertical erosion rates dz/dt (Figure 4.17).
On average, the *erosion of clay* (Figure 4.17a) is almost linearly increasing in both phases 2 and 3a. Local clay erosion (phase 2) is faster in the detailed model (36%), because the definition of the initial scenario enables to select weak sections at the inner slope with poorer clay properties, where the erosion concentrates (Table 4.6 and Table 4.7). Scour erosion of clay at the headcut base (phase 3a) is slower in the detailed model (33%), because the scour erosion is calculated with two different scour models (Sections 3.1.2.2 and 4.1.2.3). The results obtained with the detailed model should be preferred because the vertical erosion depth (dz) is directly calculated as a function of the flow in the impinging jet region (Section 4.1.2.3). In the preliminary model, on the other hand, it is only function of the undisturbed flow and of the headcut height through an empirical formula (Section 3.1.2.2).

Scour erosion of sand (Figure 4.17b) is initially very slow but soon after the first mass failure from the clay cover, it is very fast and asymptotically decreases because the shear stress at the jet impact point ($\tau_{0,jet}$) is less effective when (i) the scour hole depth (S) increases and becomes closer to the equilibrium scour hole ($S_{eq}$) and (ii) the headcut advance (dX) is the dominating headcut mechanism.

![](image)

**Figure 4.17:** Cumulated erosion depth ($\Sigma dz$) in clay cover and sand scour of the model system

(iii) **Headcut advance** (phases 3a-3b of Figure 4.6)
The growth of the headcut height ($H_{H}$) in phase 3a (Figure 4.18a), as well as the *cumulated headcut advance* ($\Sigma dX$) in both phases 3a and 3b (Figure 4.18b) are on average linear, but a stepped advance, due to cyclical mass instability at the headcut face, is clearly pronounced. The time interval ($\Delta t_{H}$) between two successive mass instabilities decreases for increasing headcut height $H_{H}$ (Figure 4.18b). Both growth of headcut height ($H_{H}$) and headcut advance ($\Sigma dX$) are much faster in the preliminary model than in the detailed model (85% and around six times faster respectively) because (i) the headcut advance is calculated with a discrete model, which slows the process, instead of a continuous averaged model.
and (ii) the selected scour erosion model (Section 4.1.2.3) predicts lower effective bottom shear stress ($\tau_{0,e}$) and vertical erosion depth $dz$ (Figure 4.17a).

The growth of the headcut width ($B_{H}$) both in phase 3a and 3b (Figure 4.18a, c) has a trend similar to the vertical erosion depth ($dz$), as a consequence of the proportionality between vertical ($dz$) and lateral ($db$) erosion through the coefficient $c_{db/dz,H}$ (Section 4.1.2.3). The final headcut width ($B_{H}$) at the end of phase 3b (Figure 4.18c), which is the width of the initial breach channel ($B_{b,0}$) is larger in the detailed model (44%), where it is calculated rather than assumed like in the preliminary model.

<table>
<thead>
<tr>
<th>Phase 3a: headcut erosion of cohesive layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Headcut height ($H_{H}$) and width ($B_{H}$)</td>
</tr>
<tr>
<td>b) Cumulated headcut advance ($\Sigma dX$)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Phase 3b: headcut erosion of sand-clay scour</th>
</tr>
</thead>
<tbody>
<tr>
<td>c) Headcut width ($B_{H}$)</td>
</tr>
<tr>
<td>d) Cumulated headcut advance ($\Sigma dX$)</td>
</tr>
</tbody>
</table>

Figure 4.18: Headcut height ($H_{H}$), headcut width ($B_{H}$) and cumulated headcut advance ($\Sigma dX$) of the model system

(iv) Breach outflow hydrograph and breach widening (phases 4-6 of Figure 4.6)

The breach outflow hydrograph ($Q_{b}(t)$) and the breach width ($B_{b}$) are qualitatively similar to those calculated by the preliminary model, because the calculation of
the core erosion and the final breach development (phases 4-6) have not changed from the preliminary to the detailed model. The peak outflow discharge ($Q_{b,p}$) and the final breach width ($B_b$) are however lower (28% and 30% respectively), because the width of the initial breach channel ($B_{b,0}$) is higher (44%).

4.2.3. Tentative model validation

The data used for the tentative validation are the same as those used for the preliminary model (Section 3.2.3.1).

Simulated versus measured data for the five selected cases in Figure 4.19, indicate a relatively good and increased level of accuracy, if compared with the preliminary model results (Table 4.11 and Figure 4.20). Results of the tentative validation generally indicate a reduced scatter between the measured and simulated breach parameters with respect to the preliminary model, both for time associated to breaching (Figure 4.19a, Figure 4.20a and Table 4.11) and breach width (Figure 4.19b, Figure 4.20b and Table 4.11):

1. **Laboratory test Nr. 10 (sand dike without clay cover):** there is no difference between the breach parameters simulated by the preliminary and the detailed model, because the modelling of the erosion of the sand core is identical in both models (Section 3.2.3.2);

2. **Laboratory test Nr. 11 (sand dike with clay cover):** the test only includes the headcut erosion of sand-clay scour and is useful to validate the calculation of the width of initial breach channel ($B_{b,0}$). The scatter between measured and simulated data is quite high (69%) for the time of core erosion ($t_{ce}$), but relatively low (9%) for the width of the initial breach channel ($B_{b,0}$). However, the detailed model improves the preliminary model where (i) the time of cover erosion ($t_{ce}$) is not defined, but identical to the time of cover failure ($t_{cf}$) and (ii) the width of the initial breach channel ($B_{b,0}$) is not calculated, but empirically assumed (Section 3.1.2.2);

3. **Polder Papendrecht (sea dike):** time of cover failure ($t_{cf}$) and time of breaching ($t_b$) are underestimated (respectively 10% and 19%), but less than in the preliminary model. The simulation is stopped when the breach width ($B_b$) is equal to its maximum admissible value, determined by the lateral heads of the dike that impose a limitation on further widening. Like for the preliminary model, final breach width ($B_b$) is therefore not compared, but imposed.

4. **Herenpolder (sea dike):** time of cover failure ($t_{cf}$) and time of breaching ($t_b$) are relatively well predicted with underestimates of respectively 4% and 6%. **Final breach width** ($B_b$) is overestimated (52%) and the scatter is in this case higher than in the preliminary model. This result can be explained in view of the uncertainties (i) in the measured data that are reported by eye-witnesses and (ii) in the model inputs relative to wave climate, grass and clay properties, that are more typical than site-specific values (Section 3.2.3.2)
5. **Zuidland (sea dike):** time of cover failure ($t_{cf}$) and time of breaching ($t_b$) are relatively well predicted with underestimates of respectively 2% and 3%. Final breach width ($B_b$) is overestimated (8%), and the scatter is reduced with respect to the preliminary model.

- **a) Time associated to breaching**

- **b) Breach width**

| Error $\varepsilon$ [%]: $\varepsilon = 100 \times \frac{|F_m - F_s|}{F_m}$ | Laboratory tests | Experienced dike failures |
|---|---|---|
| Test Nr. 10 | Test Nr. 11 | Polder Papendrecht | Herenpolder | Zuidland |
| Cover failure $t_{cf}$ [hr] | Measured | - | 0.55 | 10.50 | 19.75 | 20.50 |
| $\varepsilon_{PM}$ [1] | - | -100 | -37 | -29 | -24 |
| $\varepsilon_{DM}$ [1] | - | -69 | -10 | -4 | -2 |
| $\varepsilon_{PM} - \varepsilon_{DM}$ | - | +31 | +27 | +25 | +22 |
| Breach time $t_b$ [hr] | Measured | 0.31 | - | 22.00 | 21.50 | 22.00 |
| $\varepsilon_{PM}$ [1] | -19 | - | -42 | -27 | -26 |
| $\varepsilon_{DM}$ [1] | -19 | - | -19 | -6 | -3 |
| $\varepsilon_{PM} - \varepsilon_{DM}$ | 0 | - | +23 | +21 | +23 |
| Breach width $B_b$ [m] | Measured | 1.60 | 0.92 | 110.00 | 120.00 | 84.00 |
| $\varepsilon_{PM}$ [1] | +7 | -80 | - | +22 | +43 |
| $\varepsilon_{DM}$ [1] | +7 | -9 | - | +52 | +8 |
| $\varepsilon_{PM} - \varepsilon_{DM}$ | 0 | +71 | - | -30 | +35 |

Breach width: test Nr. 10 and Nr. 11 [cm], Polder Papendrecht, Herenpolder, Zuidland [m]
Time: test Nr. 10 and Nr. 11 [min], Polder Papendrecht, Herenpolder, Zuidland [hr]

Figure 4.19: Detailed model validation: simulated versus observed breach parameters
4.3. Capabilities and limitations of the detailed model

The detailed model, which is based on the preliminary model, represents the second step of a tiered modelling approach to dike breaching and a first attempt to achieve a fully process-oriented prediction of the breaching of sea dikes initiated by wave overtopping, overflow or a combination of both.

The detailed model (i) includes all relevant processes involved during dike breaching, (ii) introduces for the first time in a breach model, water infiltration, grass and clay instability and a simple model for breach initiation, (iii) improves the modules of the preliminary model relative to grass and clay erosion and failure and (iv) removes some simplifying assumptions (Section 4.1). Results from the detailed model are qualitatively reliable and coherent with those obtained from the preliminary model. The tentative validation against laboratory tests and
experienced dike failures shows a relatively good and encouraging agreement between measured and simulated breach parameters. Generally, the detailed model performs better than the preliminary model with a scatter generally lower than 25% (Section 4.2.3).

The model has two major limitations: (i) it is not completely process-oriented and (ii) it lacks a final validation. Further improvements and an appropriate data set for a final validation are still required, including among others:

- Complete infiltration model which directly solves Richard’s equation and a finite element model for grass and clay instability;
- Finite element model for mass instability of the breach sides (discrete model);
- A more appropriate sediment transport model for sand, which must also be valid during dike breaching;
- Physical tests on sand-clay dikes with grass cover (near full scale tests) subject to wave overtopping as primary load.

At this stage, an uncertainty analysis is important and useful for both models in order to:

- Understand how the outcomes are influenced by the input parameters;
- Achieve a more complete description of the breaching process;
- Identify and get more familiar with critical aspects.

Therefore, sensitivity and reliability analysis are applied to the entire model system (Chapter 5).
5. Uncertainties, sensitivity and reliability analyses

The model system presented in Chapters 3 and 4, is deterministic, although input parameters are stochastic. Complete outcomes from the model must therefore include quantification of uncertainties.

In this Chapter, uncertainty analysis is briefly presented, but more details are given in D’Eliso et al. (2006a, 2006b, 2007):

- Definition of uncertainties in the model system (Section 5.1);
- Sensitivity analysis: influence of single and selected sets of input parameters on the model outcomes (Section 5.2);
- Reliability analysis: influence of all input parameters on the probability distribution of the model outcomes (Section 5.3);

Finally, conclusions about uncertainties of the model outcomes are drawn (Section 5.4).

5.1. Uncertainties

Model system uncertainties are mostly due to input parameters that include dike geometry, sea, grass, clay and sand properties. Among them, material parameters play a major role (Sections 3.1.2.4 and 4.1.2.5). Model parameters that are mostly important for uncertainty analyses are the ratio between lateral and vertical erosion during headcut erosion (breaching phase 3) and breach channel erosion (breaching phases 4-6), see Figure 3.10 and Figure 4.6.

For the purpose of uncertainty analysis, uncertain parameters are assumed all normally distributed and probability distribution functions are defined by (i) mean value ($\mu$) and (ii) standard deviation ($\sigma$) or variation coefficient ($\sigma^*$):

- **Mean value:** $\mu_x = \frac{1}{N} \sum_{i=1}^{N} X_i$ [5.1]
- **Standard deviation:** $\sigma_x = \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (X_i - \mu_x)^2}$ [5.2]
• **Variation coefficient:** $\sigma_X = \frac{\sigma_X}{\mu_X}$  

Where $X$ is the variable or parameter under consideration.

All parameters, tested in the uncertainty analysis, are in the range of $\mu_X \pm 3\sigma_X$, that includes 99% of possible practicable values. A summary of ranges of parameters used in the uncertainty analyses is given in Annex B, but more details are provided in D’Eliso et al (2006a, 2006b) and Kortenhaus (2003).

### 5.2. Sensitivity analysis

Sensitivity analysis (SA) is applied to the model system to understand how uncertainties of selected uncertain parameters are transferred to the model outcomes. The analysis may have three levels of detail (Kortenhaus, 2003):

- **Level I (variation of single parameter):** only one parameter varies in the simulation, while the others are kept constant and equal to their mean value ($\mu$). It is useful to assess the importance of single parameters, select sets of parameters for Level II sensitivity analysis and provide indications on model limitations and applicability. Five realisations are used for each parameter.

- **Level II (variation of set of parameters):** a selected set of parameters varies together in the simulation, while the others are kept constant and equal to their mean value ($\mu$). It is useful to evaluate combined effects of several parameters on the outcomes. Sets of three parameters are selected where each parameter assumes three values for a total of 27 realisations used for each set of parameters.

- **Level III (variation of all parameters):** all parameters vary in the simulation. All possible combinations are considered. Because of the large number of parameters (more than 20), number of resulting realisations is too high and computational time is not affordable, even if only three values of each parameter are used. In fact, with 20 parameters, number of tests would be $3^{20} = 3.5 \cdot 10^9$.

Level I and Level II sensitivity analysis of the prototype dike (Figure 3.17) is performed. Uncertainties related to breach initiation are also analysed by varying the position of the incipient failure in the preliminary model (Section 3.1.2.1) and by defining a set of initial scenarios in the detailed model (Section 4.1.2.1).

### 5.2.1. Breach initiation

**Preliminary model (PM)**

The **location of the incipient failure** is irrelevant over the time of grass failure ($t_{gf}$). Therefore, material properties are much more relevant than the flow gradient...
at the inner slope. The weakest point along the dike profile would result from a combination of a low value of the grass cover factor ($C_f$) and a low plasticity index $I_p$ (Figure 5.1a), but in the model, only the influence of the grass cover factor is included (Section 3.1.2.1).

**Detailed model (DM)**

**Type and position of weak sections** have a major influence on grass and cover erosion and failure. Seven initial scenarios with one type of weak section dominating over the others (crack, hole, fractured clay, sand lens) and, in case of damaged grass, with different locations of the weak section (damaged grass 1, 2, 3) are defined (Figure 5.1b and D’Eliso et al., 2007).

The time of grass failure ($t_{gf}$) is not sensitive to the location, but to the type of weak section. The time of cover failure ($t_{cf}$) decreases and the initial breach channel width ($B_{b,0}$) increases as the weak section becomes closer to the dike crest, because headcut advance before complete cover failure is shorter. The time of cover failure ($t_{cf}$) is also sensitive to the type of weak section, but more as a consequence of different time of grass failure ($t_{gf}$), than of clay and headcut erosion. The initial breach channel width ($B_{b,0}$) is slightly sensitive to the type of weak section.

![Figure 5.1: Influence of grass and clay parameters on grass failure and breach initiation](image)

**5.2.2. Time of grass failure**

**Grass cover factor** ($C_f$) and **plasticity index of clay soil** ($I_p$) are the leading parameters of the time of grass failure ($t_{gf}$). Therefore, resistance of grass against erosion depends in the model on grass quality ($C_f$) and soil capacity to keep anchored the grass roots ($I_p$). The higher the two parameters are, the higher the time of grass failure ($t_{gf}$) becomes (Figure 5.1a).

Typical values of the two parameters are given in D’Eliso et al (2006b), Temple & Hanson (1994) and Temple et al. (1987).
5.2.3. Time of cover erosion and failure

Preliminary model (PM)
The time of cover erosion ($t_{ce}$) is not defined in the preliminary model, but it is identical to the time of cover failure $t_{cf}$ (Section 3.2.2).

Weight percentage of clay ($c\%$), liquid limit ($w_l$) and unconfined compressive strength (UCS) are the influencing parameters of the time of cover failure ($t_{cf}$), influencing both scour erosion and headcut advance. The cover strength is therefore modelled by the percentage of fine sediments ($c\%$), the soil plasticity ($w_l$) and the mechanical behaviour (UCS). Weight percentage of clay ($c\%$) influences both scour erosion and headcut advance. Depending on which process is dominant, opposite effects on the time of cover failure ($t_{cf}$) are observed (Figure 5.2a). Liquid limit ($w_l$) and unconfined compressive strength (UCS) only influences headcut advance with opposite effects: (i) the higher the limit liquid ($w_l$) is, the lower the time of cover failure ($t_{cf}$) becomes, but at a decreasing rate with increasing liquid limit ($w_l$) (Figure 5.2a) and (ii) the higher the unconfined compressive strength is (UCS), the higher the time of cover failure ($t_{cf}$) will be (Figure 5.2b).

Typical values of parameters are given in D’Eliso et al. (2006b), Hanson et al. (1999a), NRCS (2001) and Temple & Hanson (1994).

Detailed model (DM)
The leading parameters of time of cover erosion ($t_{ce}$) are weight percentage of clay ($c\%$), which mostly influences scour erosion in the clay cover, and clay cohesion ($c_c$), that influence headcut advance and grass and clay cover instability: the higher the weight percentage of clay ($c\%$) and the clay cohesion ($c_c$) are, the larger the time of cover erosion ($t_{ce}$) will be. However, it is more sensitive to the weight percentage of clay $c\%$ (Figure 5.3a).

Figure 5.2: Influence of clay parameters on the cover failure in the preliminary model
Chapter 5 Uncertainties, sensitivity and reliability analyses

The leading parameters of the time of cover failure \( (t_{cf}) \) are the initial saturated water front \( (z_{s,0}) \) and saturated hydraulic conductivity of clay \( (k_{s,c}) \), that may result in grass and clay cover instability (Figure 5.3b). If saturated hydraulic conductivity of clay \( (k_{s,c}) \) increases and grass or clay cover instability takes place, the time of cover failure \( (t_{cf}) \) decreases. If the initial saturated water front \( (z_{s,0}) \) increases, grass and clay cover are more prone to instability and time of cover failure \( (t_{cf}) \) may decrease due to the early instability at the saturated front (Figure 5.3b).

Typical values of parameters are given in D'Eliso et al. (2006a); Temple & Hanson (1994); Weißmann (2003). Initial saturated water front \( (z_{s,0}) \) is time-dependent and site-specific value. Suggested values are in the range of the depth of the turf base \((0.10-0.15 \text{ m})\) and \(0.40 \text{ m}\).

5.2.4. Initial breach channel width

Sediment size of sand \( (D_{50,s}) \) and tensile strength of clay \( (\sigma_c) \) are the leading parameters of the initial breach channel width \( (B_{b,0}) \), which is therefore in the model a function of the erosion resistance of sand \( (D_{50,s}) \) and the clay strength against instability \( (\sigma_c) \). Scour erosion reduces if the sediment size of sand \( (D_{50,s}) \) increases, resulting in a smaller initial breach channel width \( (B_{b,0}) \). This reduction becomes lower (i) for higher tensile strength of clay \( (\sigma_c) \) because headcut advance dominates the process and hole migration is faster than deepening and (ii) for higher sediment size of sand \( (D_{50,s}) \), because the equilibrium scour hole depth \( (S_{eq}) \) may be reached (Figure 5.4a).

The initial breach channel width \( (B_{b,0}) \) has less influence on the final breach width \( (B_b) \) than sea loads and material properties, but it is still an important parameter affecting the processes of sand erosion and final breach development (Section 4.2.2).

Figure 5.3: Influence of clay parameters on cover erosion and failure in the detailed model
Typical values of parameters are given in D'Eliso et al. (2006a) and Munkholm et al. (2002).

5.2.5. Time of dike failure, time of breaching, peak outflow discharge and final breach width

The time of dike failure ($t_{df}$) and the time of breaching ($t_b$) are here defined without considering the time of cover failure $t_{cf}$ (Figure 2.4): time of dike failure is $t_{df}-t_{cf}$, and time of breaching is $t_b-t_{cf}$. Model results (Section 3.2.2.2 and Figure 3.20) indicate that the time of dike failure ($t_{df}-t_{cf}$) is shorter than time of breach development ($t_b-t_{df}$).

**Sand sediment size** ($D_{50,s}$) and **Manning’s roughness of sand** ($n_s$) are the leading parameters of the time of breaching ($t_b-t_{cf}$), the peak outflow discharge ($Q_{b,p}$) and the final breach width ($B_b$). The higher the two parameters are, (i) the larger the time of breaching ($t_b-t_{cf}$) will be (Figure 5.4b) and (ii) the lower the peak outflow discharge $Q_{b,p}$ (Figure 5.4c) and the final breach width $B_b$ (Figure 5.4d) will also be.

Figure 5.4: Influence of clay and sand parameters on breach channel and breach flow
Typical values of the sediment size of sand ($D_{50,s}$), used as construction material for dikes, is around 0.2-0.3 mm and Manning’s roughness of sand ($n_s$) is normally around 0.2 $m^{1/3}/s$.

5.3. Reliability analysis

5.3.1. Definition and procedure

The reliability of a sea dike is its capability to operate for a given amount of time without failure. Reliability ($r$) is calculated from the probability of failure ($p_f$), which is the outcome of the reliability analysis applied to a model that simulates dike performance (Table 5.1). Three levels of reliability analysis are normally applied to engineering systems (Table 5.1).

<table>
<thead>
<tr>
<th>Table 5.1: Reliability and reliability analysis techniques</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reliability: $r = 1 - p_f$</td>
</tr>
<tr>
<td>Probability of failure: $p_f = \int_{g(x)} f_X(x) dx$</td>
</tr>
<tr>
<td>$g$: failure domain</td>
</tr>
<tr>
<td>$f_X(x)$: joint probability density function</td>
</tr>
<tr>
<td>$X = (X_1, \ldots, X_k)$ vector of random variables</td>
</tr>
<tr>
<td>(uncertain parameters of the model)</td>
</tr>
<tr>
<td>Level I (deterministic/quasi probabilistic methods):</td>
</tr>
<tr>
<td>Safety factors to apply to model outcomes are given as a standard</td>
</tr>
<tr>
<td>Level II (probabilistic methods):</td>
</tr>
<tr>
<td>Indices summarize how uncertainties transfer from uncertain parameters to model outcomes</td>
</tr>
<tr>
<td>Level III (probabilistic methods):</td>
</tr>
<tr>
<td>Full probability distribution of model outcomes is predicted</td>
</tr>
<tr>
<td>I - Numerical solutions: direct calculation of probability of failure ($p_f$)</td>
</tr>
<tr>
<td>II - Sampling techniques (e.g. Random Sampling, Latin Hypercube Sampling): a large sample (N x K) of realisations (N) of uncertain parameters (K), randomly extracted from their probability distribution functions, is used to perform a large number of simulations (N) as a sample of all possible realisations. Probability of failure ($p_f$) is approximated</td>
</tr>
</tbody>
</table>

Level III reliability analysis is time consuming, but provides more complete information on the model outcomes and is applied to the model system. Random Sampling (basic Monte Carlo) is applied to the preliminary model and Latin Hypercube Sampling to the detailed model, in order to reduce the number of realisations and computational time (Haldar & Mahadevan, 2000; Hodák & Jandora, 2004; McKay, M.D., 1992; Saliby & Pacheco, 2002):

- **Random Sampling (Monte Carlo):** it is the random extraction of values from probability distribution functions of uncertain parameters. Every new extraction is independent on the previous extraction. The sample of realisations is a set of random numbers;

- **Latin Hypercube Sampling:** it is a constrained Random Sampling scheme (McKay, M. D. et al., 1979). The range of possible values of each uncertain parameter is divided into N non-overlapping intervals. One value is randomly extracted from each interval, building a vector for each
uncertain parameter. The sample of realisations is the result of a random pairing of vectors of the uncertain parameters.

The prototype dike defined in Section 3.2 and Figure 3.17 is used for simulations. The numbers of realisations are 10000 for Monte Carlo simulations (MCs) and 2000 for Latin Hypercube Sampling simulations (LHSs), in case of both regular and irregular waves. Results are invariant to larger numbers of realisations.

5.3.2. Results and discussion

Due to the non-linearity of equations implemented in the model system, all outcomes show a non-symmetric probability distribution with a wider right tail, although all uncertain parameters are normally distributed.

5.3.2.1 Preliminary model

a) Monte Carlo simulations (MCs)

Results of Monte Carlo simulations (MCs) indicate:

- Clay cover with grass (Figure 5.5a, b): all model outcomes a have mean ($\mu$) and a standard deviation ($\sigma$) much higher in case of irregular than of regular waves. In fact, wave action is the driving force of grass and clay cover erosion and failure and the model is very sensitive to wave climate. However, the variation coefficients ($\sigma'$) are very similar, thus indicating a similar dispersion of the distribution of the outcomes around the mean value ($\mu$) and a similar relative level of uncertainties for the two models;

- Sand core (Figure 5.5c, d): all model outcomes have a mean ($\mu$) and a standard deviation ($\sigma$) which are only slightly higher for irregular than for regular waves, because wave action is less important than overflow during dike core erosion and final breach (Section 3.2.2.3).

b) Effect of sampling techniques

Comparison of Random Sampling (MCs) and Latin Hypercube Sampling (LHSs) simulations applied to the preliminary model indicates a small scatter in the model outcomes. Relative difference between the mean values ($\mu$) are within 4 % and are always higher in case of regular than irregular waves (Table 5.2).

5.3.2.2 Detailed model

Results of Latin Hypercube Sampling simulations (LHSs) indicate:

- Clay cover with grass (Figure 5.6a, b): all model outcomes have a mean ($\mu$) and a standard deviation ($\sigma$) much higher in case of irregular than of regular waves, as for the preliminary model. The time of grass erosion ($t_{eg}$) shows a bi-modal probability distribution, also evident in the time of cover
failure ($t_{gf}$), due to the implemented model for turf set-off. In fact, grass instability is not simulated as a gradual process (Sections 4.1.2.2). The failure of the grass cover occurs at the beginning of the simulation ($t_{gf} \approx 0$) due to instability or later in the simulation together with clay cover failure ($t_{gf} > 0$). However, this represents one of the major model limitations (Section 4.1.3);

- **Sand core**: the results are qualitatively very similar to those of the preliminary model (Figure 5.5c, d).

![Figure 5.5: Monte Carlo simulations (preliminary model)](image)

<table>
<thead>
<tr>
<th>Test</th>
<th>Headcut initiation $t_{hi}$ [hr]</th>
<th>Cover failure $t_{cf}$ [hr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RW</td>
<td>$\mu$ 1.60 $\sigma$ 1.68 $\sigma'$ 1.05</td>
<td>$\mu$ 1.62 $\sigma$ 1.69 $\sigma'$ 1.04</td>
</tr>
<tr>
<td>IW</td>
<td>$\mu$ 4.89 $\sigma$ 4.49 $\sigma'$ 0.92</td>
<td>$\mu$ 4.94 $\sigma$ 4.50 $\sigma'$ 0.91</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>Peak discharge $Q_{b,p}$ [m$^3$/s]</th>
<th>Final width $B_b$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RW</td>
<td>$\mu$ 1265 $\sigma$ 153.49 $\sigma'$ 0.12</td>
<td>$\mu$ 56.47 $\sigma$ 4.64 $\sigma'$ 0.08</td>
</tr>
<tr>
<td>IW</td>
<td>$\mu$ 1116 $\sigma$ 161.90 $\sigma'$ 0.15</td>
<td>$\mu$ 54.99 $\sigma$ 6.58 $\sigma'$ 0.12</td>
</tr>
</tbody>
</table>

**RW** – Regular waves, **IW** – Irregular waves

Figure 5.5: Monte Carlo simulations (preliminary model)
Table 5.2: Comparison of sampling techniques for cover failure (preliminary model)

<table>
<thead>
<tr>
<th>Test</th>
<th>Grass failure $t_{gf}$ [hr]</th>
<th>Headcut initiation $t_{hi}$ [hr]</th>
<th>Cover failure $t_{cf}$ [hr]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\mu$</td>
<td>$\sigma$</td>
<td>$\sigma'$</td>
</tr>
<tr>
<td>RW</td>
<td>3.94</td>
<td>5.43</td>
<td>1.42</td>
</tr>
<tr>
<td>IW</td>
<td>1.52</td>
<td>1.25</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Relative variation in the outcomes $\Delta F$ [%] :
- RW – Regular waves, IW – Irregular waves

Relative difference between model outcomes over time ($\Delta F$): 
$\Delta F = 100 \times \frac{|F_{MCs} - F_{LHSs}|}{F_{MCs}}$

$F_{MCs}$ - Outcome from MCs, $F_{LHSs}$ - outcomes from LHSs

a) Time of grass failure ($t_{gf}$)

b) Time of cover failure ($t_{cf}$)

Figure 5.6: Latin Hypercube Sampling simulations (detailed model)

5.3.2.3 Model system

Comparison of the outcomes of grass and clay cover failure for the preliminary and the detailed model, obtained from Latin Hypercube Sampling simulations, indicates two main differences:

- Time of grass failure ($t_{gf}$) is lower in the detailed model due to turf set-off and consequent possible grass failure at the very beginning of the simulation ($t_{gf} \approx 0$), see Figure 5.6a and Figure 5.7a;
- Time of cover failure ($t_{cf}$) has a higher mean value ($\mu$) and a higher standard deviation ($\sigma$) in the detailed model due to the different headcut erosion model implemented (Sections 3.1.2 and 4.1.2). In fact, scour
erosion is slower and the discrete headcut advance model results in lower advance rates (Section 4.2.2), see Figure 5.7b.

![Figure 5.7: Comparison between the preliminary and the detailed model (LHSs)](image)

5.3.3. Time variation of input parameters

The reliability analysis applied to the model system does not include time-variation of the input parameters, although the resistance of materials against erosion and instability decays as a result of weathering. The way probability distributions of the outcomes modifies due to time-decay of materials (mostly grass and clay), is a useful indicator of the design lifetime ($t_{dl}$) of the dike before maintenance. At present however, probability distribution of time-dependent grass and clay properties are unknown.

A deterministic relation of parameters over time is assumed including grass cover factor ($C_f$), plasticity index ($I_p$) and weight percentage of clay content ($c_p$) and assigning to them three values, namely initial value ($t = 0$), design value ($t = t_{dl}$) and residual value ($t = \infty$), see Table 5.3.

Table 5.3: Variation of selected input parameters over time

| Mean value                      | Initial ($t = 0$) | Design ($t < t_{cl}$) | Residual $t = \infty$
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Grass cover factor $C_f$ [1]</td>
<td>0.20</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>Plasticity Index $I_p$ [1]</td>
<td>12</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>Weight percentage of clay $c_p$ [%]</td>
<td>45</td>
<td>30</td>
<td>15</td>
</tr>
</tbody>
</table>

Latin Hypercube Sampling simulations of the preliminary model are performed for the three values of the selected parameters (Figure 5.8). Comparison of the mean values ($\mu$) of the outcomes shows a non-negligible variation, up to around 20%, both in case of regular (Figure 5.8a) and irregular (Figure 5.8b) waves.
Breaching of sea dikes initiated by wave overtopping

C. D’Eliso

5.4. Range and distribution of the model outcomes

The outcomes, from both preliminary and detailed model are affected by large uncertainties mainly due to the input parameters. The probability distribution of the model outcomes is not normal (Figure 5.5-Figure 5.7). Summarising the results of the sensitivity and the reliability analyses of the model system, the time associated to breaching shows larger uncertainties than the breach width and the outflow discharge (Table 5.4). In particular, for both models, the variation coefficient ($\sigma'$) is (i) around or greater than 1 for the time associated to breaching and (ii) lower than 1 for the breach width and the outflow discharge. In fact, the breach width and the outflow discharge mainly grow during breaching phase 6a (Section 3.2.2.2), while the time associated to breaching is the result of the overall process during all breaching phases. The influence of breaching initiation and of grass and clay properties on the time is the main reason of the higher values of the variation coefficient. Moreover, wave overtopping, which is also affected by large uncertainties due to the hydraulic parameters, has a different influence on the model outcomes:

- **Time associated to breaching**: all waves, although they don’t result in a flow over the dike, influence the time of the process;
• **Breach width and outflow discharge:** only waves that result in a flow over the dike, have an effect on the breach width and the outflow discharge.

Table 5.4: Range and distribution of the model system (MS) outcomes

<table>
<thead>
<tr>
<th>Outcomes</th>
<th>Range (SA)</th>
<th>Variation coefficient $\sigma'$ (MCs/LHSs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time of grass failure (PM) [hr]</td>
<td>$\mu \pm 3.00$</td>
<td>0.92</td>
</tr>
<tr>
<td>Time of grass failure (DM) [hr]</td>
<td>$\mu \pm 3.50$</td>
<td>1.25</td>
</tr>
<tr>
<td>Time of cover failure (PM) [hr]</td>
<td>$\mu \pm 2.00$</td>
<td>0.74</td>
</tr>
<tr>
<td>Time of cover failure (DM) [hr]</td>
<td>$\mu \pm 2.00$</td>
<td>0.80</td>
</tr>
<tr>
<td>Time of breaching $t_b-t_c$ (MS) [hr]</td>
<td>$\mu \pm 0.15$</td>
<td>4.17</td>
</tr>
<tr>
<td>Initial breach channel width (MS) [m]</td>
<td>$\mu \pm 1.50$</td>
<td>0.37</td>
</tr>
<tr>
<td>Peak outflow discharge (MS) [m$^3$/s]</td>
<td>$\mu \pm 300$</td>
<td>0.15</td>
</tr>
<tr>
<td>Final breach width $B_b$ (MS) [m]</td>
<td>$\mu \pm 10$</td>
<td>0.12</td>
</tr>
</tbody>
</table>

SA = Sensitivity analysis  
MCs = Monte Carlo simulations, LHSs = Latin Hypercube Sampling simulations

5.5. **Concluding remarks and implication for future research**

The overall results of the uncertainty analysis indicate that the level of uncertainties is similar for the two models (Table 5.4), because it is determined more by the input than by the model parameters. A drastic reduction of such uncertainties in the developed model system may be achieved only if specific information on the material properties and on the sea climate in front of the dike is available. Moreover, in order to reduce the uncertainties on the time associated to breaching, further research efforts must be spent in the breaching initiation model, by developing a fully predictive model instead of an initial scenario approach.
6. Summary, conclusions and recommendations

6.1. Summary of key results

The present study focuses on the breaching of sea dikes made of a sand core and a clay cover with grass initiated by wave overtopping, overflow or a combination of both (combined flow). The main objective of the study was the development of a new breach model. A new modelling strategy that consists in a model system including a preliminary and a detailed model has been proposed, where all relevant processes are accounted for. This model system represents a first step toward a full process-oriented description of dike breaching.

An overview of the work undertaken within this study is in Figure 6.1.

Figure 6.1: Work undertaken on dike breaching initiated by wave overtopping, overflow or combined flow: overview

The model system has been developed by combining and adapting existing models to describe the processes which are relevant to dike breaching. In fact, existing erosion, sediment transport and instability models were generally not...
derived and not tested for dike breaching. Basic research associated with the physics of the breaching process was not the goal of this study. The most urgent need was a model system that describes the whole breaching process. The model system should represent a process-oriented modelling framework where the specific limitations of the simulation of each single mechanism are quantified and the improvements to be made are identified.

Within this perspective, a list of processes included in the model system summarises how far the modelling framework has been advanced in this study (Table 6.1).

<table>
<thead>
<tr>
<th>PROCESS</th>
<th>AM</th>
<th>PM</th>
<th>DM</th>
<th>ACHIEVEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave overtopping</td>
<td>N</td>
<td>Y*</td>
<td>Y*</td>
<td>Both simple and process-oriented models of flow over the dike/through the breach</td>
</tr>
<tr>
<td>Main cause of breaching</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Combined flow</td>
<td>N</td>
<td>Y*</td>
<td>Y*</td>
<td>Simple wave-averaged, steady non-uniform flow model over the dike/through the breach</td>
</tr>
<tr>
<td>Main cause of breaching</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overflow</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Steady non-uniform flow model over the dike/through the breach</td>
</tr>
<tr>
<td>Determinant for breach erosion</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Infiltration</td>
<td>N</td>
<td>N</td>
<td>Y*</td>
<td>Simple models of infiltration in saturated and unsaturated soils</td>
</tr>
<tr>
<td>Main cause of breaching</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Breach initiation</td>
<td>N</td>
<td>N</td>
<td>Y*</td>
<td>Both random initial location and initial scenario approach</td>
</tr>
<tr>
<td>Estimate of warning time (tw)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grass erosion</td>
<td>N</td>
<td>Y*</td>
<td>Y*</td>
<td>Excess shear stress approach with grass parameters</td>
</tr>
<tr>
<td>Gradual failure of grass cover</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Turf set-off</td>
<td>N</td>
<td>N</td>
<td>Y*</td>
<td>Limit equilibrium approach without gradual grass failure</td>
</tr>
<tr>
<td>Instability of grass cover</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local clay erosion</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Excess shear stress approach</td>
</tr>
<tr>
<td>Gradual failure of clay cover</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Headcut erosion (clay)</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Excess shear stress approach and both continuous and discrete advance model</td>
</tr>
<tr>
<td>Gradual failure of clay cover</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Headcut erosion (sand-clay)</td>
<td>N</td>
<td>N</td>
<td>Y*</td>
<td>Non-equilibrium sediment transport and discrete advance model</td>
</tr>
<tr>
<td>Gradual formation of the initial breach channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay cover sliding or up-lift instability of clay cover</td>
<td>N</td>
<td>N</td>
<td>Y*</td>
<td>Limit equilibrium approach</td>
</tr>
<tr>
<td>Sand erosion</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Equilibrium sediment transport and simple discrete morphodynamic model</td>
</tr>
<tr>
<td>dike failure and final breach</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

AM – Available breach models, PM – Preliminary model, DM – Detailed model
Y – Included, N – Non included
* Process neglected in available breach models and included in the model system

The **preliminary model** is the first breach model that includes wave overtopping and combined flow at a sea dike with clay cover and grass, providing a first overview of the breaching process. The **detailed model** improves the preliminary model and introduces processes that were neglected. Moreover, it
builds a framework for any further modelling of dike breaching initiated by wave overtopping or combined flow.

A final validation of the model system requires specific data from well controlled near full-scale laboratory tests that are still not available. Such tests are planned at the end of this year by LWI in the Large Wave Flume of Hannover (Germany). Nevertheless, tentative validation against laboratory tests (LWI, 2005) and experienced of dike failures (1953’s flood in The Netherlands) has led to very encouraging results. In particular, the detailed model predicts both time associated to breaching and breach width with more accuracy than the preliminary model, with a relative error between simulated and measured data lower than 25%.

The uncertainties of the model outcomes which are mainly due to the uncertainties associated with input parameters are quantified with sensitivity and reliability analysis. Both models have high uncertainties in the outcomes. Probability distributions obtained from Monte Carlo and Latin Hypercube Sampling simulations have variation coefficients greater than 1 for time associated to breaching and lower than 0.20 for breach width and outflow discharge. Variation of selected material properties over the design life time of the dike to simulate the weathering of grass and clay produces a variation in the mean values of the outcomes up to 20 %.

6.2. Applicability of the proposed model system

Results achieved in the present study may be used in the engineering practice and for further research:

- **Application in engineering practice**
  The model system can be applied to sea dikes to make preliminary calculation of their resistance against breaching. In fact, conservative results are obtained and improvements in the breaching simulation are achieved from tentative validation, as intended for application in engineering practice. Nevertheless, due to lack of a final validation (Sections 3.2.3 and 4.2.3) and large uncertainties involved (Chapter 5), the outcomes should always include quantification of uncertainties using reliability analysis.
  Practical use of the model system and its outcomes includes among others: (i) integration in a fault tree of sea dikes in order to calculate the probability of flooding ($p_f$), (ii) use of the breach outflow hydrograph ($Q_b(t)$) as initial condition for flood propagation models and (iii) use of the warning time ($t_w$) to manage emergencies in case of flooding and mitigate associated damages (Section 1.1).

- **Application in research**
  The model system, together with the uncertainty analysis as demonstrated in this study, may provide valuable indications about the most urgent future research tasks and can therefore be used to support further improvements on the knowledge and the simulation of dike breaching (Section 6.3).
6.3. Future research tasks

There are still several questions to be solved which are related to the following issues:

- **Understanding of physical processes and models**
  Basic research is necessary to better understand (i) the formation of cracks during water infiltration in the dike, (ii) the gradual failure of grass due to the free surface flow and infiltration, (iii) breach initiation, (iv) the initiation of the headcut erosion, (v) the sediment transport of sand in the breach channel, (vi) the complete 3D erosion patterns for clay and sand or, alternatively, the relationship between lateral and vertical erosion which represents the key issue to reduce the 3D morphological process to a 2D problem and (vii) the instability of vertical breach slopes made of non-cohesive material. Laboratory and field tests are required to generate a consistent data set and develop interpretative models that can be used in breach models.

- **Process-oriented breach models**
  Starting from the proposed model system, a fully process-oriented model can be achieved by improving some modules to obtain: (i) 3D wave overtopping flow model, (ii) finite element model for water infiltration in the dike, (iii) detailed description of the scour hole in the cohesive layer and in sand-clay scour and (iv) prediction of the final scour at the dike base. Finally, the model system will build a more extensive prediction tool by including all other possible causes of breach initiation, like seepage and breaking wave impact, and all related failure modes resulting in a breach through the dike.

- **Detailed and final validation**
  Specific near full-scale laboratory and field tests have to be carried out to validate breach models. Tests should be performed on typical layouts of sea dikes, including grass, with wave overtopping as primary load, by measuring all breaching parameters (time, outflow and breach size) and identifying transition between process phases.

Some of the identified future research tasks already represent the focus of research projects within the International Community, due to the importance of the topic for the assessment and management of coastal flood risks.
## List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tbody>
<tr>
<td>$1:m$</td>
<td>Outer slope</td>
<td></td>
</tr>
<tr>
<td>$1:n$</td>
<td>Inner slope</td>
<td></td>
</tr>
<tr>
<td>$A_{b}$</td>
<td>Breach cross-section area</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>$A_{p}$</td>
<td>Polder area</td>
<td>$[m^2]$</td>
</tr>
<tr>
<td>$B_{b}$</td>
<td>Width of breach channel</td>
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</tr>
<tr>
<td>$B_{b,0}$</td>
<td>Initial width of breach channel</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$B_{d}$</td>
<td>Width of dike crest</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$B_{h}$</td>
<td>Headcut width</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$B_{o}$</td>
<td>Width of the overtopping or combined flow tongue</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$c$</td>
<td>Wave celerity/Flow velocity</td>
<td>$[m/s]$</td>
</tr>
<tr>
<td>$C$</td>
<td>Courant number for numerical instability</td>
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<tr>
<td>$C_{f}$</td>
<td>Friction coefficient</td>
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<tr>
<td>$c_{%}$</td>
<td>Weight percentage of clay in cohesive soil (clay cover)</td>
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<tr>
<td>$c_{c}$</td>
<td>Clay cohesion</td>
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<td>$C_{d}$</td>
<td>Jet diffusion coefficient</td>
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<tr>
<td>$c_{db/dz,b}$</td>
<td>Ratio between lateral and vertical erosion in the breach channel</td>
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<td>Ratio between lateral and vertical erosion in the headcut scour hole</td>
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<td>$C_{f}$</td>
<td>Grass cover factor</td>
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<td>Grass root cohesion</td>
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<td>$C_{i}$</td>
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<td>Sediment concentration</td>
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<td>$c_{\text{tot}}$</td>
<td>Total cohesion (grass root and clay)</td>
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<td>$D$</td>
<td>Expected damages and losses</td>
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<td>$D_{50}$</td>
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<tr>
<td>$D_{75,c}$</td>
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<td>$db$</td>
<td>Lateral erosion depth</td>
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<td>$dx$</td>
<td>Longitudinal erosion depth</td>
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<tr>
<td>$dz_{c}$</td>
<td>Cumulated vertical erosion depth</td>
<td>$[m]$</td>
</tr>
</tbody>
</table>
| Symbol | Description | Unit/

<p>| Wave frequency | [1/\text{s}] |
| Local Froude number of flow | [1] |
| Acceleration due to gravity | [\text{m/s}^2] |
| Flow depth | [\text{m}] |
| Wave height | [\text{m}] |
| Backwater flow depth in the headcut scour hole | [\text{m}] |
| Height of the breach channel | [\text{m}] |
| Initial height of the breach channel | [\text{m}] |
| Combined flow depth | [\text{m}] |
| Critical flow depth | [\text{m}] |
| Dike height | [\text{m}] |
| Position of phreatic line in the dike | [\text{m}] |
| Headcut height | [\text{m}] |
| Initial headcut height | [\text{m}] |
| Overflow head | [\text{m}] |
| Backwater level | [\text{m}] |
| Initial backwater level | [\text{m}] |
| Significant wave height at dike toe | [\text{m}] |
| Plasticity index of clay | [%] |
| Energy slope | [1] |
| Erodibility coefficient of clay | [\text{m}^3/(\text{N}\cdot\text{s})] |
| Headcut erodibility coefficient of clay | [1] |
| Saturated hydraulic conductivity | [\text{m/s}] |
| Saturated hydraulic conductivity of clay | [\text{m/s}] |
| Saturated hydraulic conductivity of clay with grass | [\text{m/s}] |
| Saturated hydraulic conductivity of sand | [\text{m/s}] |
| Length of failing block of soil in turf set-off, cover sliding or up-lift | [\text{m}] |
| Deep water wave length | [\text{m}] |
| Stem length | [\text{m}] |
| Mean high tide water level | [\text{m}] |
| Mean water level in the sea | [\text{m}] |
| Manning’s roughness of clay | [\text{m}^{1/3}\cdot\text{s}] |
| Number of time steps within a wave cycle | [1] |
| Manning’s roughness of sand | [\text{m}^{1/3}\cdot\text{s}] |
| Total Manning’s roughness (grass and clay) | [\text{m}^{1/3}\cdot\text{s}] |
| Probability of dike failure | [1/\text{yr}] |
| Pegelnull: reference for water level measurements (Germany) | [\text{m}] |
| Porosity of sand | [1] |
| Water discharge per unit width | [\text{m}^3/\text{sm}] |
| Combined outflow hydrograph/discharge | [\text{m}^3/\text{s}] |
| Combined flow discharge from the breach | [\text{m}^3/\text{s}] |
| Peak outflow discharge | [\text{m}^3/\text{s}] |
| Combined flow discharge per unit width | [\text{m}^3/\text{sm}] |
| Equivalent overflow discharge per unit width | [\text{m}^3/\text{sm}] |
| Bed volumetric sediment transport discharge per unit width | [\text{m}^3/\text{sm}] |
| Suspended volumetric sediment transport discharge per unit width | [\text{m}^3/\text{sm}] |
| Total volumetric sediment transport discharge per unit width | [\text{m}^3/\text{sm}] |
| Total volumetric sediment transport discharge | [\text{m}^3/\text{s}] |</p>
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<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_{sw,b}</td>
<td>Breach outflow discharge of water-sediment mixture</td>
<td>[m$^3$/s]</td>
</tr>
<tr>
<td>R</td>
<td>Coastal flood risk</td>
<td>[€/yr]</td>
</tr>
<tr>
<td>R_b</td>
<td>Hydraulic radius of the breach</td>
<td>[m]</td>
</tr>
<tr>
<td>R_t</td>
<td>Wave run-up</td>
<td>[m]</td>
</tr>
<tr>
<td>S</td>
<td>Scour hole depth</td>
<td>[m]</td>
</tr>
<tr>
<td>S(f)</td>
<td>Wave energy spectrum</td>
<td>[m$^2$s]</td>
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<tr>
<td>S_c</td>
<td>Thickness of the clay cover</td>
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<td>S_{eq}</td>
<td>Equilibrium scour hole depth</td>
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<td>S_Q</td>
<td>Backwater level coefficient</td>
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<td>SWL</td>
<td>Still water level in the sea</td>
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<tr>
<td>t</td>
<td>Time</td>
<td>[s]</td>
</tr>
<tr>
<td>T</td>
<td>Wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>T</td>
<td>Water temperature</td>
<td>[°C]</td>
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<tr>
<td>t_0</td>
<td>Time of initiation of wave overtopping and overflow</td>
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<tr>
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<td>Time of breaching</td>
<td>[s]</td>
</tr>
<tr>
<td>t_{ce}</td>
<td>Time of cover erosion</td>
<td>[s]</td>
</tr>
<tr>
<td>t_{cf}</td>
<td>Time of cover failure</td>
<td>[s]</td>
</tr>
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<td>t_d</td>
<td>Time of breach development</td>
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</tr>
<tr>
<td>t_{df}</td>
<td>Time of dike failure</td>
<td>[s]</td>
</tr>
<tr>
<td>t_f</td>
<td>Time of breach formation</td>
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<tr>
<td>t_{gf}</td>
<td>Time of grass failure</td>
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</tr>
<tr>
<td>t_{hi}</td>
<td>Time of headcut initiation</td>
<td>[s]</td>
</tr>
<tr>
<td>t_i</td>
<td>Time of breach initiation</td>
<td>[s]</td>
</tr>
<tr>
<td>T_P</td>
<td>Peak wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>t_w</td>
<td>Warning time</td>
<td>[s]</td>
</tr>
<tr>
<td>u</td>
<td>Suction pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>u_a</td>
<td>Pore-air pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined compressive strength of clay</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>u_w</td>
<td>Pore-water pressure</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>v</td>
<td>Flow velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>V_O</td>
<td>Volume of water released in the polder area</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>V_{O,tot}</td>
<td>Total volume of water released in the polder area</td>
<td>[m$^3$]</td>
</tr>
<tr>
<td>w</td>
<td>Water content</td>
<td>[%]</td>
</tr>
<tr>
<td>w_l</td>
<td>Liquid limit</td>
<td>[%]</td>
</tr>
<tr>
<td>w_s</td>
<td>Settling velocity of sand</td>
<td>[m/s]</td>
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<tr>
<td>X_p</td>
<td>Jet entry point</td>
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</tr>
<tr>
<td>y_c</td>
<td>Breach channel axis</td>
<td>[m]</td>
</tr>
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<td>Z</td>
<td>Thickness of the failing of soil in turf set-off, cover sliding or up-lift</td>
<td>[m]</td>
</tr>
<tr>
<td>Z_b</td>
<td>Breach bottom elevation</td>
<td>[m]</td>
</tr>
<tr>
<td>z_s</td>
<td>Saturated water front</td>
<td>[m]</td>
</tr>
<tr>
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<td>Averaged saturated water front at the inner slope</td>
<td>[m]</td>
</tr>
<tr>
<td>z_w</td>
<td>Infiltration (unsaturated) water front</td>
<td>[m]</td>
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<tr>
<td>α</td>
<td>Outer slope angle (plane XZ)</td>
<td>[rad]</td>
</tr>
<tr>
<td>β</td>
<td>Inner slope angle (plane XZ)</td>
<td>[rad]</td>
</tr>
<tr>
<td>γ</td>
<td>Breach slope angle (plane YZ)</td>
<td>[rad]</td>
</tr>
<tr>
<td>δ</td>
<td>Breach channel curvature (plane XY)</td>
<td>[rad]</td>
</tr>
</tbody>
</table>
Δ Relative density of sand particles [1]
Δt Time step [s]
Δx Grid spacing [m]
Δz_g Erosion depth at grass failure [m]
η Water surface [m]
θ Volumetric water content [1]
θ_i Initial volumetric water content [1]
θ_r Residual volumetric water content [1]
θ_s Saturated volumetric water content [1]
μ_{Comb} Discharge coefficient for combined flow [1]
μ_{Over} Discharge coefficient for overflow [1]
μ_X Mean value of the parameter X -
v Kinematic viscosity of water [m²/s]
ξ_d Surf similarity parameter [1]
ρ Material density [kg/m³]
ρ_c Density of clay [kg/m³]
ρ_{c,d} Dry density of clay [kg/m³]
ρ_{cw} Density of water-clay sediment mixture [kg/m³]
ρ_{g,s} Density of sand grains [kg/m³]
ρ_s Density of sand [kg/m³]
ρ_{s,d} Dry density of sand [kg/m³]
ρ_w Density of water [kg/m³]
σ_c Tensile strength of clay [N/m²]
σ_g Averaged tensile strength of the grass root [N/m²]
σ_X Standard deviation of the parameter X -
σ_{X'} Variation coefficient of the parameter X -
τ Soil shear stress [N/m²]
τ_0 Bottom shear stress [N/m²]
τ_{0,cr} Critical shear stress [N/m²]
τ_{0,e} Effective bottom shear stress [N/m²]
φ_{0_s} Angle of repose of sand [rad]
φ_c Angle of friction of clay [rad]
φ_s Angle of friction of sand [rad]
χ_{jet} Angle of jet impact [rad]
Ψ Suction head - Flow head [m]
Ψ_b Bubbling suction head [m]
Ω_b Wet section of the breach [m²]
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Breaching of sea dikes initiated by wave overtopping


Broich, K. (2004): Description of the parameter model DEICH_P. *4th Workshop of the IMPACT Project, Final Meeting, 3-5th November*, Zaragoza, Spain, pp. 16.


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Harris, G.W.; Wagner, D.A. (1967): Outflow from breached earth dams. *Department of Civil Engineering, University of Utah*, Salt Lake City, UT.


Breaching of sea dikes initiated by wave overtopping

C. D’Eliso


Annex A: Available breach models

List and classification
<table>
<thead>
<tr>
<th>MODEL</th>
<th>YEAR</th>
<th>TYPE</th>
<th>APPLICATION</th>
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<tr>
<td>CRISTOFANO (Cristofano, 1965)</td>
<td>1965</td>
<td>PB-A</td>
<td>CD</td>
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<tr>
<td>HARRIS – WAGNER (Harris &amp; Wagner, 1967)</td>
<td>1967</td>
<td>PB-P</td>
<td>HD-FPD</td>
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<td>THIRRIOT (Thirriot, 1975)</td>
<td>1975</td>
<td>NA-P</td>
<td>D</td>
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<td>PONCE – TSIVOGLOU (Ponce &amp; Tsivoglou, 1981)</td>
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<td>PB-1DN</td>
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<td>LOU (Lou, 1981)</td>
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<td>1984</td>
<td>NPB-E</td>
<td>D</td>
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<td>DAMBRK (Fread, 1984)</td>
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<td>SMPDBK (Wetmore &amp; Fread, 1984) simplified version of DAMBRK</td>
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<td>NWS BREACH (Fread, 1988)</td>
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<td>BLED (Singh &amp; Scarlatos, 1987)</td>
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<td>DEICH P (Brechteler &amp; Broich, 1991)</td>
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<td>FROEHLICH (Froehlich, 1995)</td>
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<td>FPD</td>
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<td>BREIS (Visser, 1998b)</td>
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<td>PB-P</td>
<td>SAND DIKE</td>
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<td>DEICH N2 (Broich, 2004)</td>
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<td>KRAUS (Kraus &amp; Hayashi, 2005)</td>
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<td>SPB-A/N</td>
<td>CB</td>
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<td>MIKE 11, Dam Break Module (DHI software)</td>
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<td>FLOW SIM 1 and FLOW SIM 2 (Bodine, undated)</td>
<td>undated</td>
<td>SPB-P</td>
<td>D</td>
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<tr>
<td>RENARD Electricité de France/ RUPRO Cemagref, FR</td>
<td>NA</td>
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NA – Non Available information

NPB – Non Physically-based
SPB – Semi Physically-based
PB – Physically-based
E – Empirical
A – Analytical
P – Parametric
1DN/2DN – 1D/2D Numerical

D – Dams
HD – Homogeneous Dams
FPD – Fuse Plug Dams
CD – Cohesive Dams
NCD – Non-cohesive Dams
LD – Landslide Dams
ES – Earth Spillways
CB – Coastal Barriers
Annex B: Prototype dike

Input parameters used in the preliminary and in the detailed model, including uncertainties
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<th>Symbol-Unit</th>
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<td>Δz_g [%L_s]</td>
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<td>c_g [N/m²]</td>
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<td>2.8-17.2</td>
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<td>w_l [%]</td>
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<td>16.1-53.9</td>
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<td>UCS [N/m²]</td>
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<td>c_d [N/m²]</td>
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<td>500</td>
<td>0.20</td>
<td>800-3200</td>
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<td>Initial headcut height</td>
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<td>H_{H,0} [m]</td>
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<td>0.02</td>
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<td>Friction angle</td>
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<td>φ_d [deg]</td>
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<td>17.5-32.5</td>
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SA = Sensitivity Analysis  
PM = Preliminary Model  
RA = Reliability Analysis  
DM = Detailed Model  
Continued …
# SAND PARAMETERS (DIKE CORE)

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<th>( \sigma )</th>
<th>( \alpha )</th>
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<td>PM-DM</td>
<td>( \rho_{d,i} ) [kg/m³]</td>
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<td>( \rho_g ) [kg/m³]</td>
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<td>Pore size distribution index</td>
<td>DM</td>
<td>( N_{bc,s} ) [1]</td>
<td>N</td>
<td>0.57</td>
<td>3.530</td>
<td>0.10</td>
<td>0.06-1.08</td>
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<td>Parameter ( n ) (van Genuchten curve)</td>
<td>DM</td>
<td>( n_{vG,s} ) [1]</td>
<td>N</td>
<td>2.68</td>
<td>0.057</td>
<td>0.10</td>
<td>0.27-5.09</td>
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<tr>
<td>Parameter ( \alpha ) (van Genuchten curve)</td>
<td>DM</td>
<td>( \alpha_{vG,s} ) [1/cm]</td>
<td>N</td>
<td>0.15</td>
<td>0.265</td>
<td>0.10</td>
<td>0.01-0.28</td>
<td>X</td>
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SA = Sensitivity Analysis  
PM = Preliminary Model  
RA = Reliability Analysis  
DM = Detailed Model
Annex C: Photos’ references
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<th>Figure</th>
<th>Location</th>
<th>Reference</th>
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<tr>
<td>Figure 1.3a</td>
<td>Nössedeich near Morsum (1981)</td>
<td>Sönnichsen U., Moseberg J. (1997): Wenn die Deiche brechen. Sturmfluten und Küstenschutz an der schleswig-holsteinischen Westküste und in Hamburg. Husum Druck und Verlagsgesellschaft</td>
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<td>Figure 1.3b</td>
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<td>Petersen M., Rohde H. (1991): Sturmflut. Die großen Fluten an den Küsten Schleswig Holsteins und in der Elbe, Wachholtzverlag</td>
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<td>Augustenbroden Ostseite Jadebusen (1962)</td>
<td>Die Küste, Heft 1 (1962)</td>
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<td>Figure 2.3g</td>
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<td>Sönnichsen U., Moseberg J. (1997): Wenn die Deiche brechen. Sturmfluten und Küstenschutz an der schleswig-holsteinischen Westküste und in Hamburg. Husum Druck und Verlagsgesellschaft</td>
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